END-PLATE CONNECTIONS AND ANALYSIS OF SEMI-RIGID STEEL FRAMES

by

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To my mother and to the memory of my father

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DECLARATION

I hereby declare that the work embodied in this thesis is the result of my own investigations except where specific reference has been made to the work of others. No part of it has been submitted to any university for a degree, diploma or other qualification.

SUMMARY

The thesis examines the behaviour of end-plate connections and the related topic of analysis of unbraced semi-rigid steel plane frames with criteria for their design.

The static moment-rotation behaviour is investigated for connections consisting of bolted flexible end-plates, flush and extended end-plates, between I-section members, where the beams frame into the flanges of the columns. Attention is focussed on moment-rotation characteristics as this is the most important influence on the response of either individual members or complete frames. The non-linear nature of these characteristics is identified and methods of representing moment-rotation curves for subsequent use in analytical procedures are presented. A data base for such types of connection is created.

An established computer program for second-order frame analysis has been extended to frames with semi-rigid connections. Successive estimates are made of the secant stiffness of these connections to represent their effect on frame behaviour. The analysis program has been used to study the effects of semi-rigid joints on frame behaviour.

Studies have been carried out to extend the Merchant-Rankine formula for the assessment of the ultimate load of frames with semi-rigid joints over the application range of which was until now restricted to frames with rigid joints.

A parameter entitled 'degree of flexibility' is introduced as a measure of the effect of semi-rigid joints on the stiffness of the frame. Within the limits of the study, it is demonstrated that the second-order effects will not be significant if the semi-rigid elastic critical load exceeds ten times the design load, and the degree of flexibility is less than 50%. This last requirement was satisfied by extended end-plate beam-to-column connections. It has also been found that, under combined loading, the serviceability limit on sway is likely to control design, rather than ultimate strength.

Based on experimental and theoretical studies, it is recommended that the BS5950 simplified method should permit 20% end-restraint, which would improve significantly the attractiveness of the method.

NOTATION

Additional symbols are defined locally for special purposes

A	Cross-sectional area
a	Stiffness paramater (EA/L)
a	Beam flexibility (L/EI)
B_p	Plate width
b	Stiffness parameter ($12\ EI\ \varphi_2/L^3$)
C_1, C_2	Functions of the connection parameters
c	Direction cosine
D_b	Beam depth
d	Stiffness parameter ($-\ 6\ EI\ \varphi_2/L^2$)
E	Young's modulus of elasticity
e	Stiffness parameter ($4\ EI\ \varphi_3/L$)
F	Yield stress
f	Stiffness parameter ($2\ EI\ \varphi_4/L$)
G	Horizontal gauge distance
H	Horizontal wind load
h	Storey height
I_b	Beam second moment of area
I_c	Column second moment of area
i	Joint number
j	Joint number
$\mathbf{K}$	Overall stiffness matrix
K	Member stiffness matrix
k	Secant stiffness
$\mathbf{L}$	Load vector

L	Length of beam
L_b	Vertical distance between extreme bolt's centre
L_p	Plate length (depth)
M	General term for bending moment
M_c	Moment capacity of connection
M_e	Beam end moment
M_f	Free moment
M_h	Semi-rigid hinge moment ($M_h = - k\Phi$)
M_p	Plastic moment of resistance
M_s	Span moment
M_y	Yield moment
P	Vertical pitch distance
P_f	Proof load of bolts
P_l	Preload force of bolts
P_i	Connection parameter
s	Direction sine
T_{fc}	Column flange thickness
T_p	End-plate thickness
T_{wb}	Beam web thickness
T_s	Column web stiffener thickness
$\mathbf{X}$	Joint displacement vector
X	x-direction of overall coordinate system
x	Connection flexibility (C k)
Y	y-direction of overall coordinate system
y	Column flexibility
V, W, P	Vertical loading
w, q, q_o	Uniformly distributed load
Z	Connection factor representing initial tangent flexibility section modulus

$\alpha_{i,k}$	Regression coefficients
α_i, β_i	Coefficients
γ	Partial safety factor
δ	Displacement
ν	Poisson ratio of material
λ_{cr}	Lowest elastic critical load
λ_p	Rigid-plastic collapse load
λ_f	Elastic-plastic failure load
$\lambda_{f,mr}$	Merchant-Rankine failure load
$\lambda_{f,mrw}$	Merchant-Rankine-Wood failure load (modified Merchant-Rankine failure load)
θ	Rigid-joint rotation
Φ	Semi-rigid connection rotation
φ_1 - φ_s	Livesley's stability functions

CHAPTER I

INTRODUCTION

1.1 Preamble

A steel frame as a plane or spatial structural system consists of linear members, such as beams and columns, jointed together by connections. These joints between members play an important role. From an economic point of view the costs for design and fabrication form a considerable part of the total costs. From a structural point of view the properties of the joint essentially influence the response of the structure to actions. Traditionally, in the design and analysis of steel frames it is assumed that all structural beam-to-column connections behave either as:

i) simply pinned, which implies that no moment will be transmitted between the beam and the column and thus the connection is only capable of transmitting shear and axial force. As far as rotation is concerned, the beam and the column that are jointed together by a pin will behave independently;

ii) fully rigid, which implies that no relative rotation will occur between the adjoining members and the beam end-moment is transmitted to the column. The angle between the beam and the column remains unchanged as the frame deforms.

Existing methods of design and analysis of steel frames still firmly rely on these simplified idealised models, despite the fact that it has been recognised for more than half a century [1.1, 1.2] that the connection behaviour lies somewhere between the two extreme idealisations. Thus the design methods of the structure is not fully based on the actual load-deformation characteristic of the joint. This is due to a number of factors, two of which are the relative complexity of the necessary design calculations and lack of comprehensive information on the performance of the full range of modem day connections. This has resulted in a rapid growth of investigation on this topic, leading to continuous innovations in the analysis and design of steel frames and the testing of beam-to-column connections.

Experiments carried out during the last decades have shown that the behaviour of real bolted connections is neither rigid nor pinned; rather, they possess some degree of rotational restraint which depends on the type of connection used. The term "Semi-rigid" is used to describe such connections.

In addition to the classification by rigidity, beam-to-column connections may be classified as well by strength with respect to the design moment resistance. Under such classification, beam-to-column connections behave either as:

i) nominally pinned connections where the connection should be able to transmit the forces calculated in design, without developing significant moments which might adversely affect members of the structure;

ii) full-strength connections, in which the design resistance of a full strength connection should be at least equal to that of the member connected;

iii) partial-strength connections, in which the design resistance of the connection may be less than that of the members connected.

Thus a connection can be semi-rigid or rigid and at the same time be either partial-strength or full-strength.

This thesis presents further research and developments in behaviour and modelling of end-plate connections (flexible, flush and extended end-plate) and analysis of flexibly connected steel plane frames. Before embarking on the scope and objective of the present work it is necessary to define some characteristics of semi-rigid connections and concepts of semi-rigid analysis.

1.2 Origin and background of steel frames with semi-rigid connections.

Beam-to-column connections are vital components in steel framed structures and have been a subject of research since the early part of this century, as witnessed by Wilson and Moore [1.1]. This first experimental investigation was carried out in 1917 to determine the rigidity of riveted joints in steel structures. In the 1930's, more testing of beam-to-column connections were conducted in Britain [1.2], Canada [1.3] and the United States [1.4] and has been necessary ever since to understand the behaviour of the joint and its effect on the overall structure. Since then numerous tests on riveted, bolted and welded connections have been reported.

Between 1929 and 1936, the Steel Structures Research Committee [1.2] investigated several aspects of riveted and bolted connections, to establish a basis for analysis of steel framed structures.

In 1934, in a second report for the Committee [1.2], Batho and Rowan proposed a graphical method, known as the "Beam-Line Method", for predicting the end-restraint provided by a connection for which the experimentally obtained moment-rotation curve was known.

The work undertaken by the Committee [1.2] formed the basis of BS 449's provisions [1.5] to incorporate semi-rigid end restraint into the analysis and design of steel framed structures.

From the early investigations into connection behaviour it was realised that economy was possible by reducing the moment and deflection at the centre of a beam. These offer reductions in the sections of uniform members at no additional cost in fabrication. This is in contrast to the use of rigid connections which are unpopular because they are expensive to fabricate.

One of the outcomes of the early investigations [1.2, 1.6] was a saving of 15-20% of the weight of beams in a frame achieved by taking advantage of end-restraint as opposed to assuming simple connections.

The benefit of connection stiffness in reducing the mid-span moment of a beam subjected to a uniformly distributed load with different end behaviour, is shown in Figure 1.1. For a pin ended beam, the mid-span moment, M_f, is critical and there is no moment, M_e, at the supports (Figure 1.1a). However, for fully fixed ends, the beam end-moment, M_e, is critical (Figure 1.1c). If semi-rigid connections are assumed, the end-moments and the mid-span moment can be balanced, resulting in a lower critical moment and hence in economies in design as previously mentioned. A further benefit is the reduction of mid-span deflections due to the inherent stiffness of the joint. The ratio of mid-span deflection for simply supported and fully fixed beams is 5:1. A reduction in beam deflection as a result of semi-rigid action may permit a reduction in beam weight. Therefore, by utilising the inherent strength and stiffness of connections which are currently considered to be pinned, a semi-rigid approach promises economy in the design of members with no additional cost in jointing.

The influence of semi-rigid end restraint on column behaviour has only been investigated more recently than that on beams, an early work being that of De Falco and Marino [1.7]. A modification of the relative stiffnesses of beams with semi-rigid connections, to be used in determining effective lengths of columns was presented by Driscoll [1.8] and much work has been carried out ever since. Since effective length factors can be reduced due to the additional inherent restraint provided by the connections, another possible economy lies in column design [1.8, 1.9].

The influence of end restraint on column stability has been examined by several researchers [1.10]. Modest moment-rotation stiffness may enable slender columns to carry significantly greater loads than those indicated by designs which assume pinned ends. In many cases the inherent joint stiffness may be accounted for in column design by estimating an effective length based on an assessment of the degree of restraint provided at the column's ends [1.11]. Test results [1.12] suggest that effective lengths shorter than those currently adopted can be justified in "simple" braced frames.

For beam-to-column connections, the most useful characteristic to define is the moment-rotation, M-Φ, characteristic. In order to incorporate the effect of semi-rigid joint behaviour in any sort of frame analysis, a knowledge of the M-Φ response, or at least the ability to approximate the key parts of the M-Φ curve adequately, is a prerequisite for carrying out such analysis.

1.3 Types of beam-to-column connections:

For steel frame construction, it is common practice to connect the beams to columns on site. A variety of connections are used. These connections usually involve some combination of bolting and welding. Normally it is preferred that bolts should be used on site and welding in the fabrication shop. Three classes of bolt are in common use in the U.K., these are:

i) ISO metric black hexagon head bolts to BS 4190 (available in three strength grades),

ii) ISO metric precision hexagon head bolts to BS 3692 (available in ten strength grade), and

iii) High strength friction grip bolts to BS 4395.

For convenience these are usually referred to 'black', 'precision' and 'HSFG' respectively. Grade 4.6 black bolts and Grade 8.8 precision bolts are the norm.

With the introduction of high strength friction grip bolts (HSFG) in the early fifties in the USA and Europe, both the HSFG and black bolts have been used increasingly on site since the fasteners require only limited specialist skills and have advantages in assisting erection, alignment, etc. Bolts have replaced rivets which are now not in use on modern steel structures. The reasons for the switch from rivets to HSFG bolts are discussed by Munse et al. [1.13].

HSFG fasteners consist of high tensile bolts and nuts with hardened steel washers. The bolts are tightened to a predetermined shank tension and in shear connections the load is transferred between the plies by friction. The bolts do not act in shear or bearing as in ordinary bolted joints. When placed in a tension joint, the connected parts cannot separate until the torque's shank tension is exceeded by the applied tension. Installation is therefore a more critical operation and can increase site erection costs. BS 4604 gives the necessary requirements to achieve a high degree of tightening control.

Comparative tests on grade 8.8 and HSFG bolts in tension and shear [1.14] have found that the 8.8 bolt is not significantly weaker in tension than the HSFG bolt, provided grade 10 nuts are used to avoid premature stripping of the threads in the nuts. The high strength bolts behave in a similar manner at failure to HSFG bolts.

In recent years, bolts grade 8.8 have been used very extensively and consequently the popularity of HSFG bolts have been decreased substantially. Therefore savings in material costs and field labour costs can be achieved by using 8.8 bolts rather than using HSFG bolts if preloading is not required.

Welding on site is generally avoided if possible and any welding required for a connection is done in the shop, under better quality control conditions than can be achieved on site. Site-welded moment connections are not popular due to a number of reasons, three of which are, (i) expense, (ii) erection difficulties in welded steel frames where it is not possible to pack the joints together to facilitate a fully aligned structure, and (iii) in many locations, limitation of welding due to climatic conditions. Bolted connections may be more flexible than fully welded joints, thus accentuating the need for semi-rigid design methods.

The ten connection types shown in Figure 1.2 were the subject of a survey [1.15] of beam-to-column connections used by the construction industry in the U.K. The survey revealed that the most frequently used connections are angle web cleat, flush end-plate and extended end-plate, being used by 87%, 96% and 93% of replies respectively. The header plate (i.e. flexible end-plate) connection, according to the European Convention for Constructional Steelwork (ECCS) [1.16] is also a frequently used type of joint. The main forms of beam-to-column connections assume the use of I-sections for both types of member.

1.4 Moment-rotation characteristic and joint classification:

Typical moment-rotation, M-Φ, curves for the most commonly used beam-to-column connections are shown in Figure 1.3. Such diagrams have been presented in many papers on semi-rigid joints. In this graphical format, the vertical (M) axis represents the perfectly rigid connection, and the horizontal (Φ) axis represents the perfectly pinned connection. Real connections fall within the quadrant between the two axes. The closer the curve is to the horizontal axis, the more flexible is the connection whilst those which approach the vertical axis are performing almost as a rigid joint. At this stage it is appropriate to define some of the characteristics of semi-rigid connections.

i) The moment-rotation curve is defined as a curve expressing the moment, M, transmitted by the connection, from beam to column, as a function of the relative rotation, Φ, of the two members fastened by the connection (i.e. of the elastic lines of the connected members at their point of intersection).

ii) The connection rotation may be easily seen by considering Figure 1.4. The connection on the top is rigid while that shown on the bottom is semi-rigid. In the case of the rigid connection, under the application of load, both the column and the end of the beam will rotate through an angle θ. However, in the semi-rigid connection, the column will rotate through an angle θ while the beam rotates through an angle φ. The relative rotation, $\varphi - \theta = \Phi$ is defined as the connection rotation.

$$\Phi_{connection} = \varphi - \theta \tag{1.1}$$

The measurement of joint rotation has been discussed at length in references [1.17, 1.18]. Therefore, care must be taken to ensure that correct connection rotation values are measured in experiments when comparing experimental moment-rotation curves obtained from different sources. An approximation for modelling the real behaviour of a beam-to-column connection is shown in Figure 1.5 where the connection is represented as a rotational spring connecting the centrelines of the column and the connected beam at their point of intersection [1.19].

iii) The moment resistance, i.e. moment capacity of the connection, M_c, is equal to the peak value of the moment-rotation characteristic [1.19], (see Figure 1.6).

iv) The rotation capacity, Φ_c, of a beam-to-column connection shall be taken as the rotation achieved at the moment resistance of the connection [1.19], (see Figure 1.6).

v) Each connection has its own moment-rotation curve characteristic and the slope, k, of the M-Φ curve is a measure of the rigidity or stiffness of the connection at any particular value of rotation. A typical moment-rotation curve is shown in Figure 1.6. Three stiffness values can be identified with any M-Φ curve:

a) the initial stiffness,

b) the tangent stiffness at any point, and

c) the secant stiffness.

Any of the three may be utilised in the analysis, so long as it is consistent with the overall technique used. This will be discussed further in Chapter 5 of this thesis which concerns the analysis of semi-rigidly connected steel frames.

All common forms of beam-to-column connection exhibit non-linear M-Φ behaviour over almost the entire range of loading and the connection stiffness decreases as rotational deformation increases (see Figure 1.3). The reasons for the latter statement are discussed by Lewitt et al [1.20].

The classification of a connection's rigidity depends on both the connection type and the connection parameters. As cited earlier, all joints in structural steelwork may be classified as pinned, semi-rigid or rigid [1.21]. This is a convenient classification from the point of view of both joint design and structural frame analysis. The crucial characteristic that determines the classification is the moment-rotation characteristic. In references [1.9, 1.22, 1.23] a stiffness classification of connections is presented.. The quadrant which limits the two extremes idealisation is divided into 4 categories: pinned, flexible, stiff and rigid, based on connection stiffness [1.22]. The most commonly used connections were grouped as stiff or as flexible connections. This classification does not present any specified boundary limit to separate the different categories.

In Eurocode 3 [1.19] the classification of beam-to-column connections are based on either the moment resistance (clause 6.9.6.3), or on the rotational stiffness of the connection (clause 6.9.6.2). The classification with respect to rotational stiffness, in a non-dimensionalised $\bar{m}$ - $\bar{\Phi}$ diagram, is illustrated in Figure 1.7 for connections in braced frames and in Figure 1.8 for unbraced frames. Connections which are classified as rigid or nominally pinned, may optionally be treated as semi-rigid.

1.5 Modelling of the moment-rotation curves:

The main problem with the analysis of semi-rigid connections is the large number of design variables. Each connection has its own characteristic moment-rotation curve which depends on geometrical size (i.e. connection dimensions, beam and column dimensions); material properties, bolt sizes, types and locations; bolt pretension...etc. A change of one component will lead to a change of moment-rotation curve characteristic. This curve must be either determined experimentally or analytically, before one can proceed to the analysis of semi-rigid steel frames. The production of moment-rotation curves from actual tests is time-consuming and expensive to cover the complete practical range for all types of semi-rigid connections. However, the other economical alternative is to develop an M-Φ prediction equation which can simulate the behaviour of the semi-rigid connection based on a data base covering a sufficient range of values or based on mechanical models.

This basic need for the analysis of flexibly connected frames is in effect highly complex, because of the large number of joint parameters, both geometrical and mechanical, which occur to the joint behaviour. Several representations of the M-Φ curves have been developed over the years.

In 1934, in a second report for the Steel Structure Research Committee [1.2], Baker proposed the first attempt at fitting a mathematical representation to a connection's M-Φ curve. A similar attempt was made in 1936 by Rathburn [1.4] in the USA. These were based on the assumption that the joints could be modelled by a linearly elastic moment-rotation characteristic, thus making the overall structural response linearly elastic. The equation of such line may be expressed as:

$$Z = \frac{\Phi}{M} \tag{1.2}$$

in which Z is the initial tangent flexibility. The value of Z is the inverse of the initial slope of the moment-rotation curve at zero moment. This linear assumption of joint behaviour was incorporated into early methods of frame analysis with semi-rigid end restraint, in particular into the slope-deflection and moment distribution methods [1.2, 1.4]. These methods will be discussed at length in Chapter 5.

This representation makes the method only strictly applicable for very low values of rotation and becomes increasingly inaccurate as the moment increases, as shown in Figure 1.9. To overcome this, in the 1970s a bilinear model was proposed in which the initial slope of the M-Φ line was replaced by a shallower line at a certain transition moment, M_T (see Figure 1.9). Such models were developed by Lionberger and Weaver [1.24] and Romstad and Submarian [1.25]. Such an approach, apparently rough, presents the advantage of drawing attention only to two basic parameters and not all the moment-rotation curve. This bilinear curve is composed of two linear parts. The first part exactly coincides with the initial stiffness, k_1, of the connection which was used in the linear model while the definition of the second part was set either at zero slope (i.e. curve c Figure 1.9) as proposed by several investigators [1.26, 1.27, 1.28] or at a reduced slope (i.e. curve b Figure 1.9), k_2, as proposed in references [1.24, 1.25]. The transition moment, M_T, is set to the ultimate moment capacity of the connection when the slope of the second part of the curve is set to zero. Following this, a trilinear model was developed by Moncarz and Gerstle [1.29] and the quadrilinear model by Melchers and Kaur [1.30] to represent the experimental curve. Note the use of a trilinear representation requires specification of two moment values and three stiffnesses.

As far as it is known, the first attempt to fit a nonlinear M-Φ curve using the experimental data was by Batho and Lash [1.2] presented in the final report of the Steel Structures Research Committee in 1936. This curve fitting was represented by a power function of the form:

$$M = C\,\Phi^{0.412} \tag{1.3}$$

where C is a constant dependent on the type and dimension of the connection. The first attempt to fit a polynomial series to M-Φ data was by Sommer [1.31] in 1969. The work was then extended for different types of connections by Frye and Morris [1.32].

In 1982, Jones et al [1.33] utilised a cubic B-spline to fit moment-rotation curves to experimental data. The model divided the experimental data into a number of smaller ranges within which a cubic function was fitted with first and second derivatives continuous across the sub-divisions. Although the method produces accurate fits, it would require the storing of an extremely large amount of information if it were incorporated into a structural analysis program.

In addition to the connection models described in the preceding sections, numerous researchers have developed their own special models that were used to describe the nonlinear behaviour of their own connection tests. Basically, all the research into modelling of moment-rotation curves falls into one of the following categories:

i) *Empirical curve fitting model*:

This is based on gathering of data on moment-rotation curves of connections and employing curve fitting and regression techniques to develop prediction equations. This procedure generally employs size and shape parameters. Curve fitting techniques are commonly employed to convert experimental data into curves which are represented algebraically. Such techniques have been used extensively over the last two decades. These powerful methods of curve fitting of experimental data are suitable to represent at high accuracy level all the range of practical interest, of the connection behaviour. The available prediction equations however show a degree of precision very dependent on the connection type and on the range examined.

ii) *Analytical models*:

These semi-empirical models are developed in two separate stages. Firstly, simplified models for the main parameters defining the M-Φ curve, such as initial elastic stiffness, ultimate moment capacity, major sources of connection deformation etc, are determined. In the second phase, a curve fitting technique is used to develop the full M-Φ relationship.

iii) *Mechanical models*:

Unlike the previous described models generated purely (i) or partially (ii) from empirical curve fitting, the mechanical models are based on the consideration of the deformation behaviour of the connection elements. Mechanical models with complete interaction existing between the individual components constituting the connection are very difficult to develop. So far, such approach has been used by a very few investigators for joints with rather simple physical behaviour.

Nethercot and Zandonini [1.34] recognise these models as the most suitable in principle, provided that a knowledge of the load-deformaton curve of the principal components is available.

iv) *Finite element models*:

With the increasing use of high speed computers, the finite element method of analysis has developed considerably and make it possible to study complete problems in structural analysis.

Recently the finite element method has been used to study the overall behaviour of semi-rigid connections, in which elastic, elastic-plastic and collapse behaviour of the connection can be achieved. Such models are very difficult, complex and require the modelling of a very large number of parameters such as: the nonlinear element material capability; various components of the members and of the connection; weld representation; bolt behaviour, preload and hole clearance; initial imperfections; interaction forces, etc. As far as it is known, the first attempt to use finite element method for steel joints was introduced to welded beam-to-column connections by Bose et al [1.35]. The web was considered as the critical component of the connection and studied in isolation as a plate strength problem using a load pattern simulating the moment and shear transmitted by the beam. An incremental procedure was used to solve the problem.

Although, considerable work has been carried out since then, the numerical analysis of bolted connections using finite element methods is still limited and does not yet seem suitable for bolted beam-to-column connections until further refinements are made.

1.6 Aim of the study:

The aim of the work reported in this thesis is (i) to provide an in-depth study of moment-rotation characteristics and corresponding parameters for the most commonly used beam-to-column connections in steel construction, (ii) to develop a computer program for second-order elastic frame analysis incorporating semi-rigid connections with criteria for design and (iii) to investigate a simple semi-rigid design method of steel frames, with a critical review of the redistribution rules in BS 5950.

Design models for the M-Φ relationships of connections can only be based on a systematic evaluation of test information. Considerable amount of testing of beam-to-column connections has been carried out over the years mainly in Europe, USA and Canada. Several researchers have published papers discussing the behaviour and the influences of connection rigidity on steel frame structures for all connection types. Several attempts [1.23, 1.36, 1.37] have been made to review the experimental M-Φ data in order that direct comparisons are possible between the different reported works and for use in future studies of joint and frame behaviour. In 1983, Goverdhan [1.36] collected a large number of test data and provided M-Φ curves, numeric data and available prediction equations of moment-rotation relationships for the seven types of commonly used beam-to-column connections. Additionally Nethercot [1.23] conducted a literature survey for the period 1915-1985. In his paper, he reviewed test data for steel beam-to-column connections and the corresponding curve representations. He summarised the available test data and supplied comprehensive tables containing details of the 12 main types of connection between I-section members. More recently, in 1986 Chen and Kishi [1.37] produced a comprehensive collection of data on joint tests similar to Goverdhan's work. References [1.36, 1.37] provide digitised information for seven types of commonly used beam-to-column connections, from which M-Φ curves can be plotted. Almost without exception, bolting was used for the part of the connection that would be made on site.

In the study reported herein, the static moment-rotation behaviour was investigated for connections consisting of bolted flexible end-plate, flush and extended end-plates between I-section members where the beams frame into the flanges of the columns (i.e. major axis connections only). Despite the previous investigations [1.23, 1.36, 1.37] that have reviewed the experimental M-Φ data, the present author felt it necessary to check, complete and update the information on these three particular connection types. The Kishi and Chen [1.37] data bank which was published in 1986 covers only 104 experiments for these three particular types of connections.

Although the three investigations [1.23, 1.36, 1.37] have been carried out in the past, the literature review shows that comparatively little is known about matters such as bolt tightening procedure, bolt preload, yield stress of the material used, failure load and failure mode of the connections, etc. In this study, for each experimental datum not only moment-rotation characteristics has been included but also all parameters used in beam-to-column connections which could be applied to formulate the prediction equations.

The principle objectives of this research are as follows:

i) To obtain moment-rotation characteristics for all the available experimental data and present M-Φ data in a digitised form.

ii) To identify those aspects of typical major axis connections which have significant influence on the moment-rotation characteristics.

iii) To review and compare experimental data against the available prediction equations.

iv) To develop an empirical model to represent the behaviour of such connections for use in subsequent research.

v) To undertake sections (i), (ii) and (iii) for flexible, flush and extended end-plate connections. Step (iv) is carried only for flush end-plate connections.

vi) To develop a computer program for second-order elastic analysis of semi-rigidly connected steel frames.

vii) To study the criteria for neglect of second order effects using the developed computer program.

viii) To carry out pilot studies on a) an approximate method for ultimate load analysis and b) significance of sway as a design criteria.

ix) To investigate the sensitivity of the results of frame analysis of complete frameworks, including second order effects, to M-Φ characteristics.

x) To investigate a simple semi-rigid design method of steel frames with a critical examination of the redistribution rules on BS 5950 for semi-rigid design.

1.7 Outline of the thesis:

An extensive study has been made of available experimental data on semi-rigid connections, and on their analytical representation. Part A of this thesis is based on a review of all the earlier work relevant to the modelling of moment-rotation curves and to the available experimental data to evaluate the static moment-rotation performance of semi-rigid beam-to-column connections consisting of flexible end-plate, flush and extended end-plate. The Kishi and Chen [1.37] data bank which was published in 1986 covers only 104 experiments, however, the author's survey includes 296 experiments where the complete M-Φ curves were quantified (where possible) for connections of varying geometry with their corresponding parameters. From the experimental test data base, the influence of joint parameters such as end-plate size, number, size, type and preload force of bolts, local stiffening arrangements etc., on the initial stiffness, moment capacity, failure moment, failure mode of the connection and on the M-Φ behaviour, are identified by a parametric analysis. A collection of the available moment-rotation prediction equations and models for moment capacity of the connection are presented. The available moment-rotation curves are compared against the moment-rotation equations. The validity, advantages, disadvantages of these models are described and recommendations made regarding their application and their use for analysis and design of semi-rigid steel frames. The moment-rotation curves presented by the original investigators have been digitised with a reasonably accurate approximation of the experimental data where the numerical form of the M-Φ data could not be obtained by the author, to form the data base of steel beam-to-column connections. Flexible end-plate connections are presented in Chapter 2. Flush end-plate connections to stiffened and unstiffened columns are considered in Chapter 3. Chapter 4 is devoted to the extended end-plate connections to stiffened and unstiffened columns. Particular attention is paid to the flush end-plate joint; it was found that no existing analytical representations applied to this kind of joint. The author has therefore developed an empirical model to represent the connection behaviour.

The prediction equation for end-plate connections presented in EC3 [1.19] based on Zoetemeijer's work [1.38, 1.39, 1.40, 1.41, 1.42, 1.43, 1.44], developed in the Netherlands over a number of years, is not included in this thesis. The reader should refer to recent paper of Moore [1.45] where the model is presented in full and the work on which it is based is identified and discussed.

Part B of this thesis deals with the analysis of semi-rigid steel frames and criteria for design. Background and methods of analysis of semi-rigidly connected steel frames are given in Chapter 5. Studies on single subframes where column far ends are either fixed or pinned with header plate connections are carried out using beam line method. Boundary limits of column to beam flexibility ratio and connection to beam flexibility ratio for a feasible error in beam design moment in order to ignore column flexibility are investigated. A nonlinear elastic computer program for the analysis of frames with any combination of pinned, rigid or semi-rigid connections is developed by the present author and presented in the same chapter.

The effects of semi-rigid joints on frame behaviour are studied in Chapter 6. This includes studies of the extent and significance of sway and distribution of bending moment from joint flexibility for unbraced frames. The effect of M-Φ relationship on the elastic analysis results of steel frame is also investigated. An approximate method for ultimate load analysis and the significance of sway as a design criteria for semi-rigid unbraced frames are determined.

Chapter 7 investigates a simple semi-rigid design method of steel frames with a critical examination of the redistribution rules in BS 5950 for semi-rigid design.

Appendices A, B and C cover the digitised moment-rotation data base for flexible end-plate, flush and extended end-plate connections respectively. Each experimental datum has included not only moment-rotation characteristics but also all parameters used in beam-to-column connections which could be applied to formulate the prediction equations. In this study, the test identification number used by the original investigators has been kept to avoid any possible confusion.

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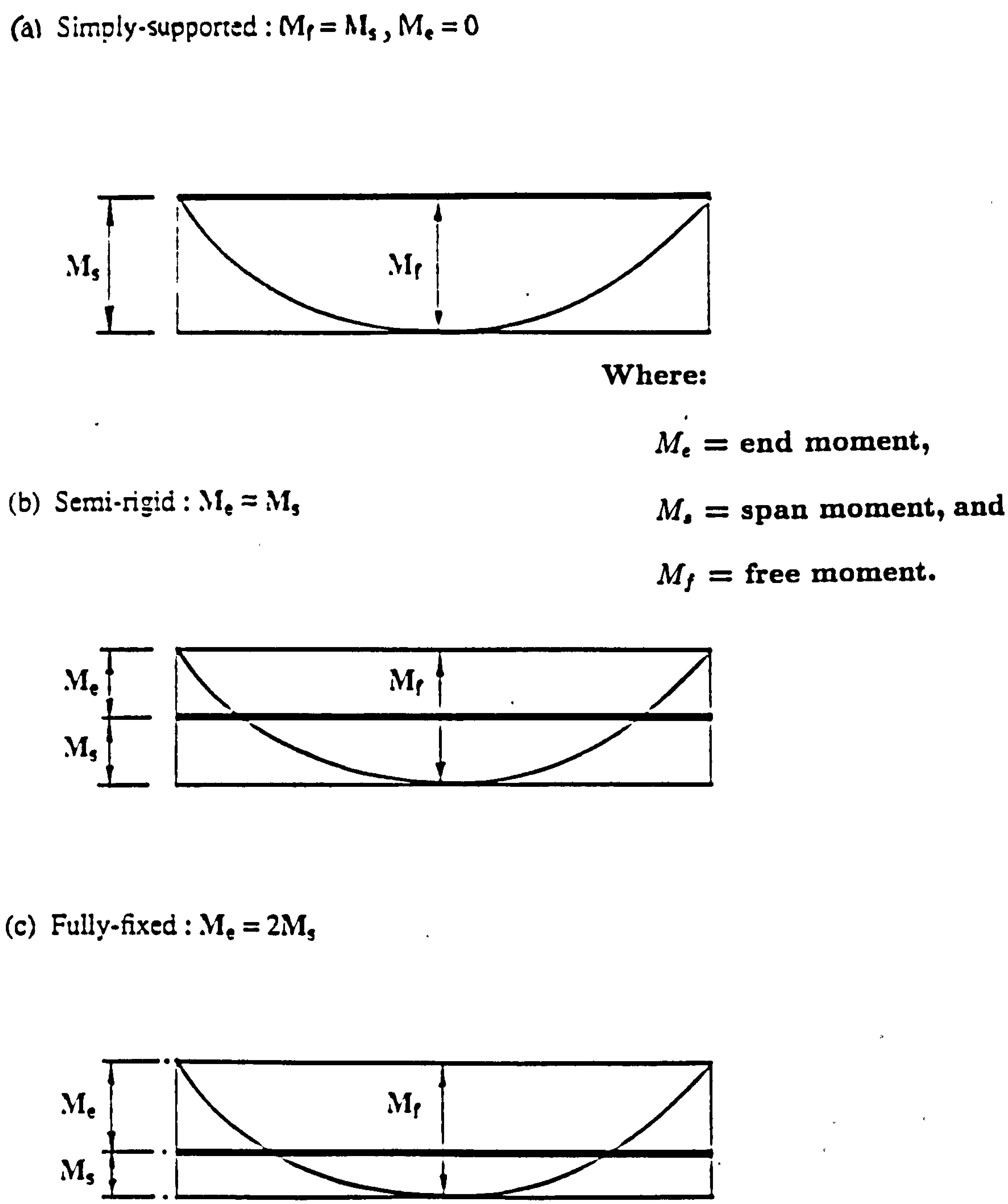


Figure 1.1 Classes of beam-to-column connection

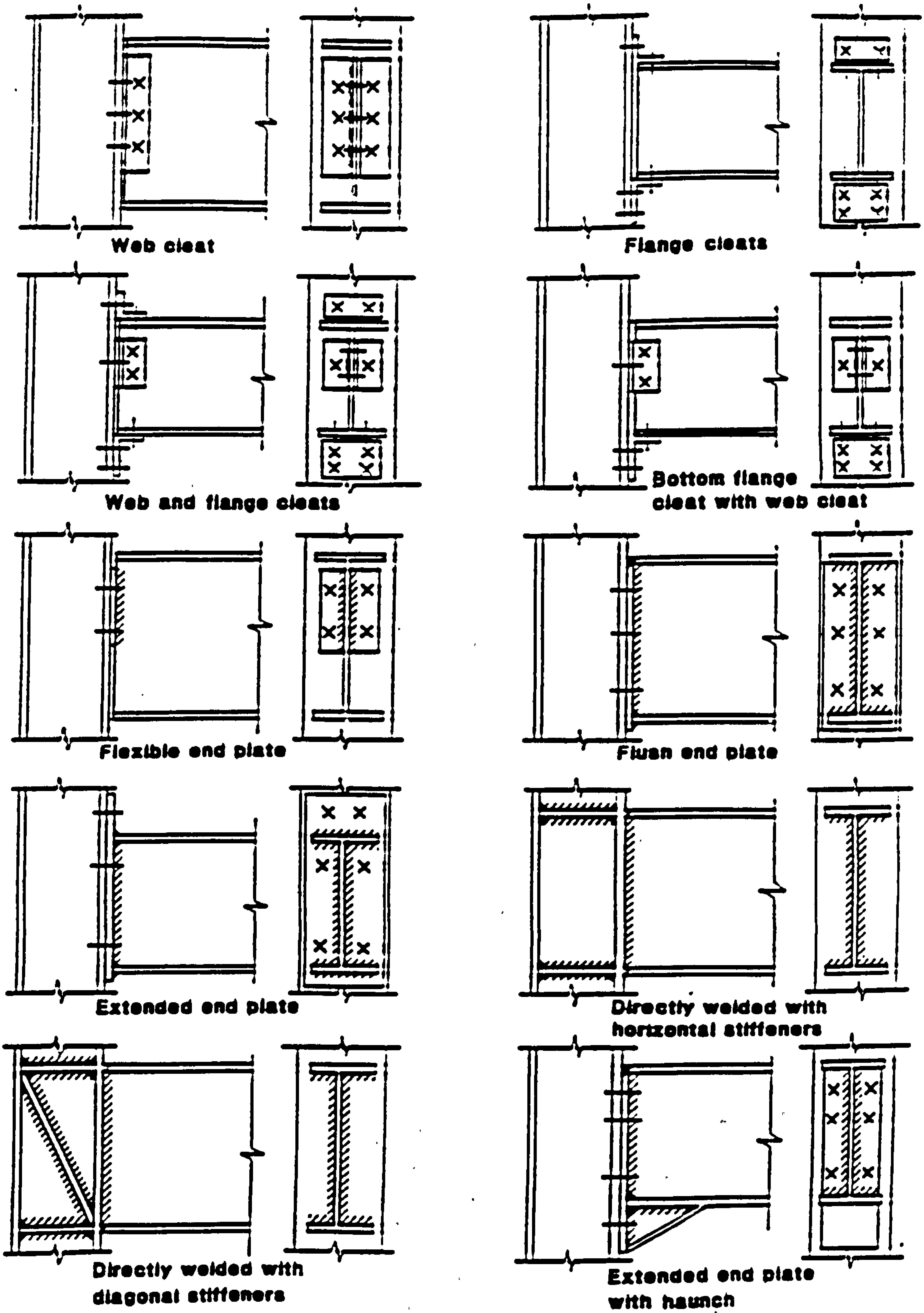


Figure 1.2 Common beam-to-column connection types.

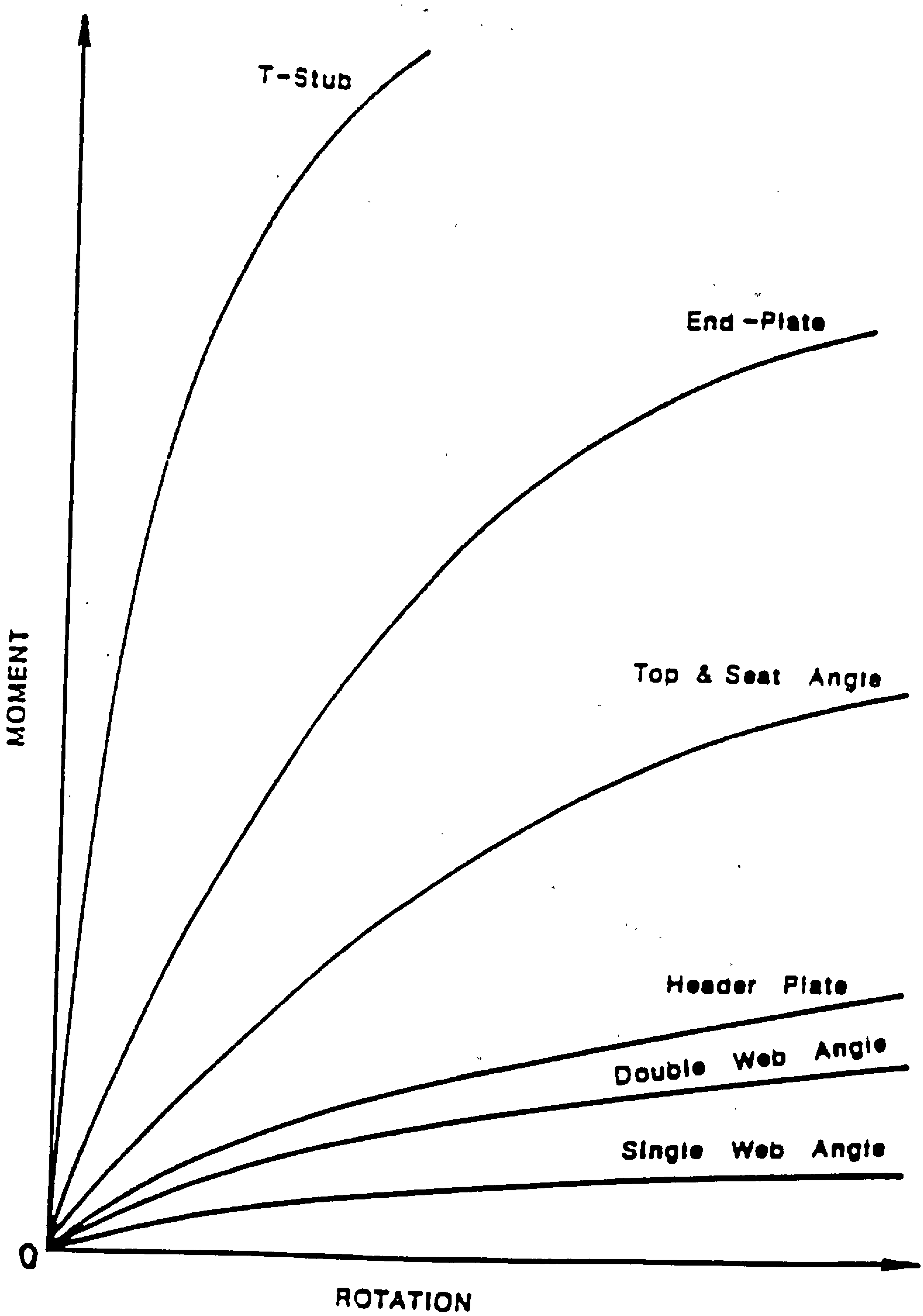
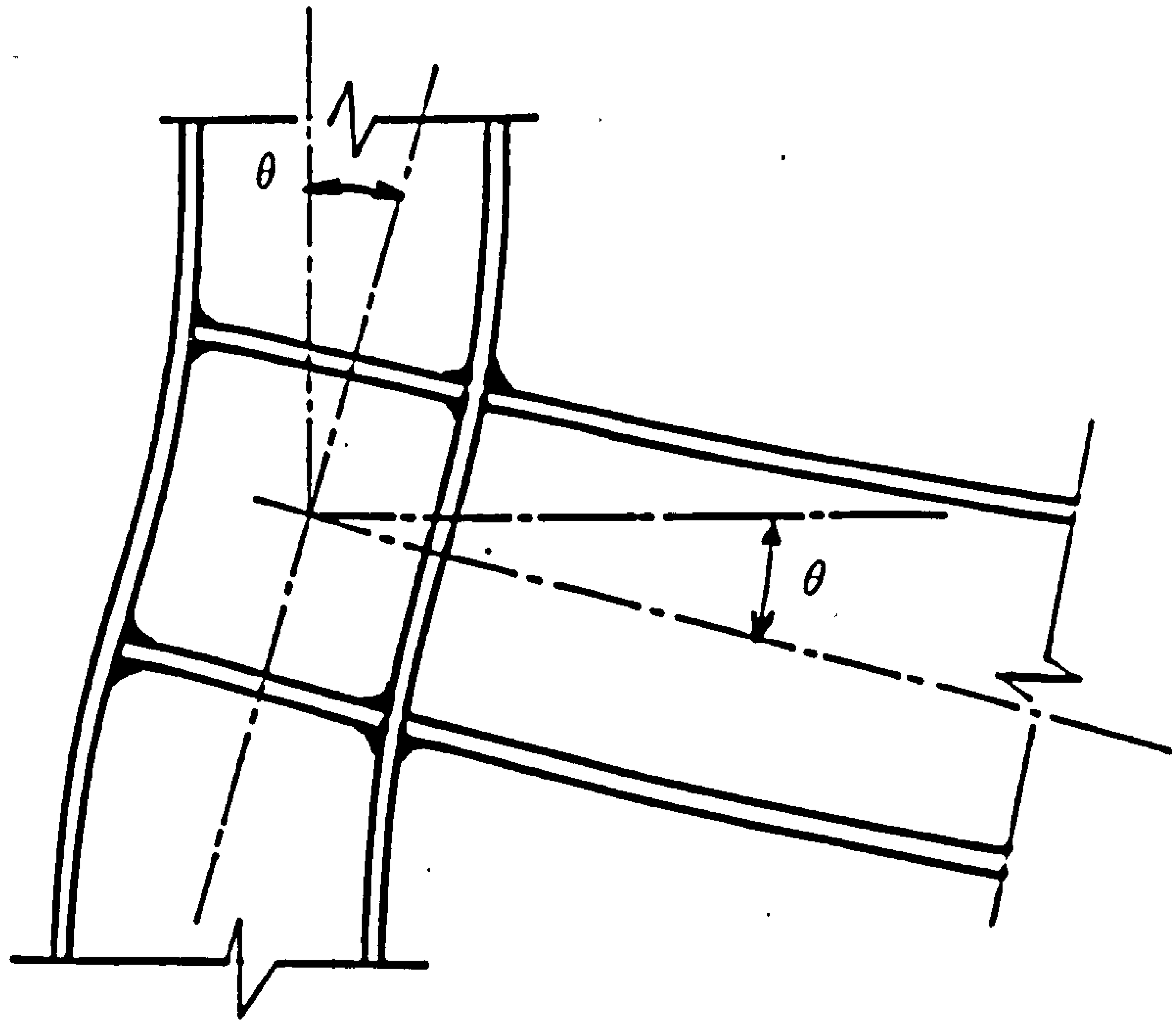
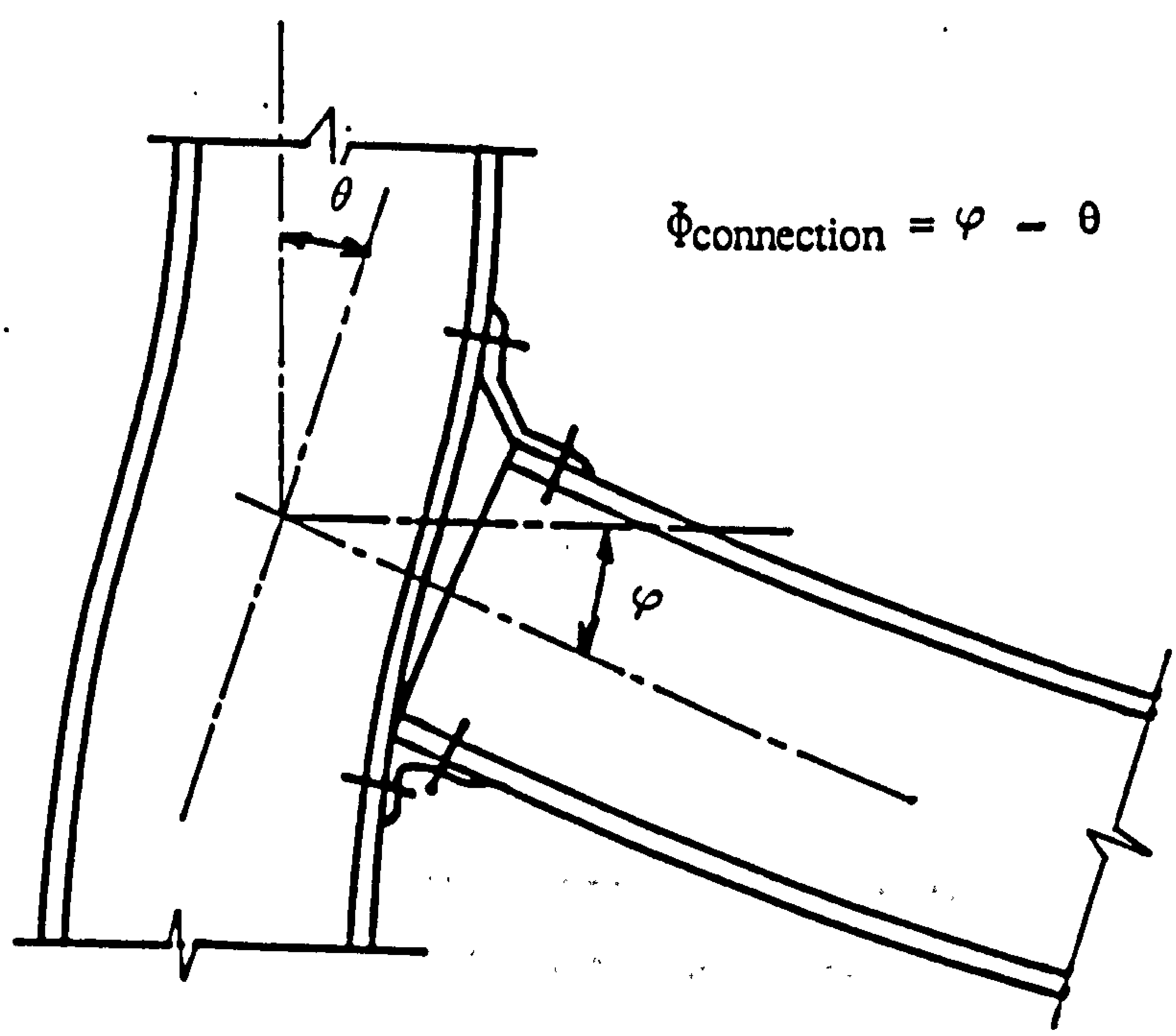


Figure 1.3 Connection moment-rotation curves.



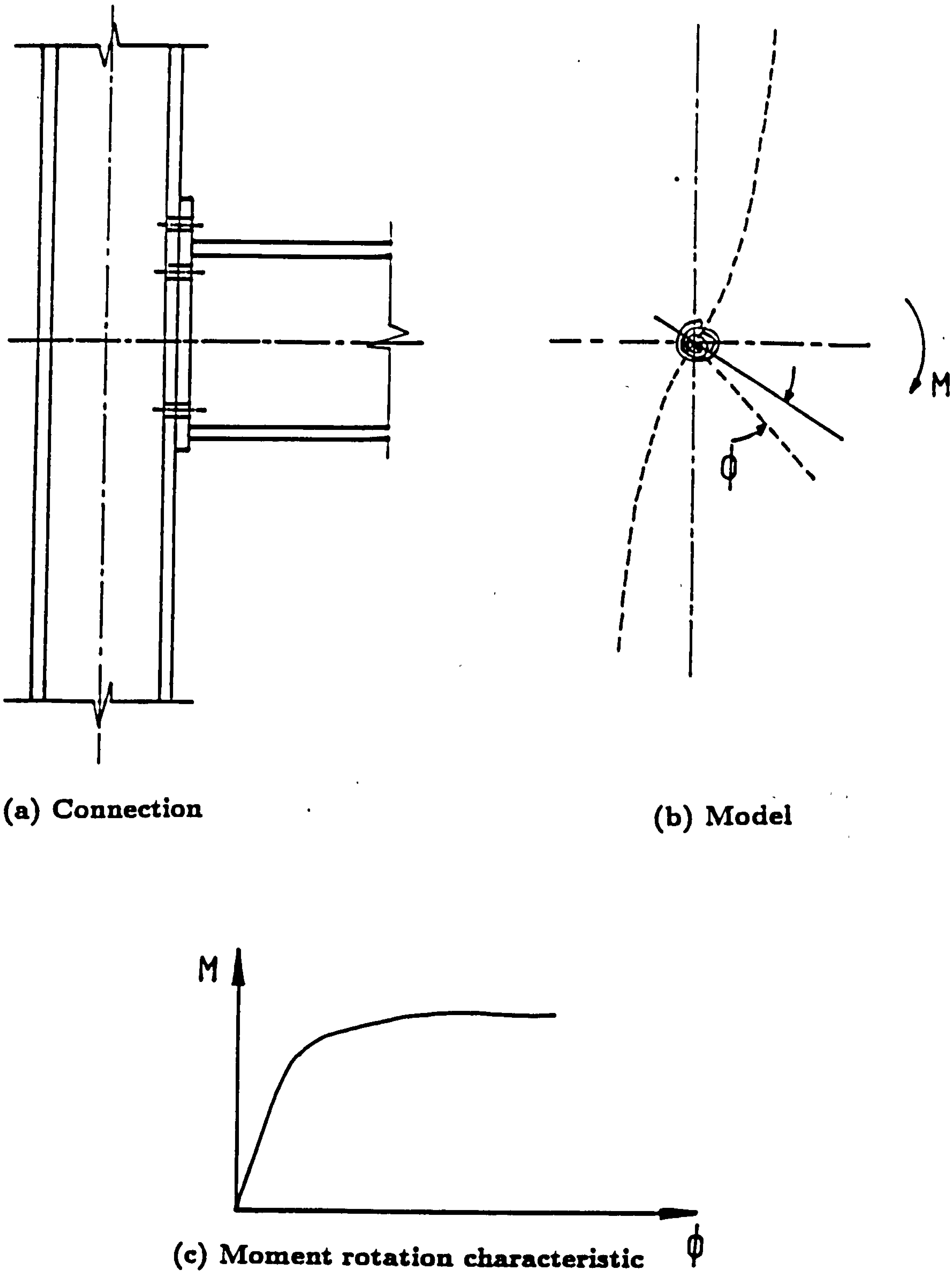
(a) Rigid



(b) Semi-rigid

Figure 1.4 Definition of connection rotation.

(a) Rigid and (b) Semi-rigid connection.



(a) Connection

(b) Model

(c) Moment rotation characteristic

Figure 1.5 Modelling a connection as a rotational spring.

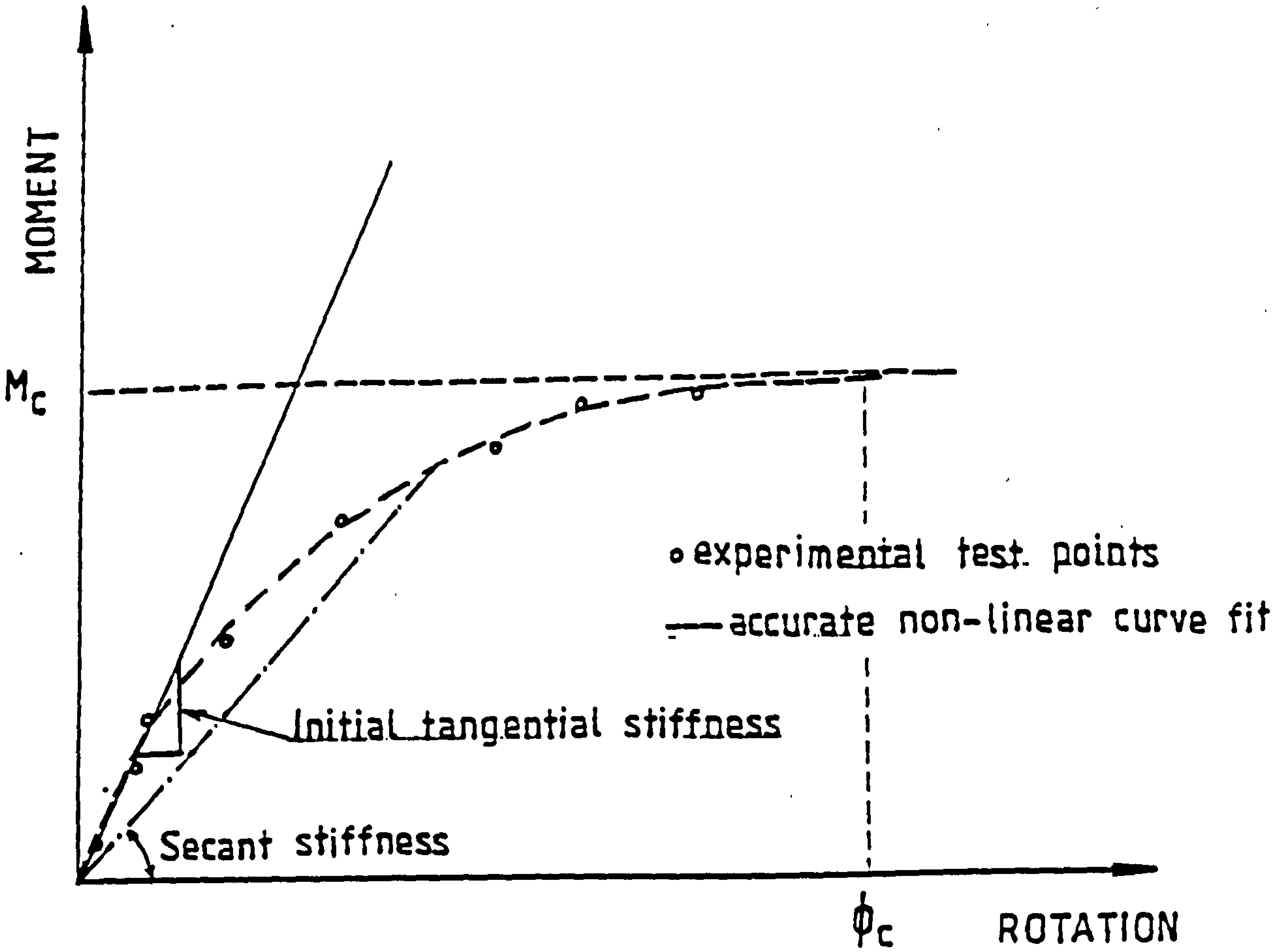
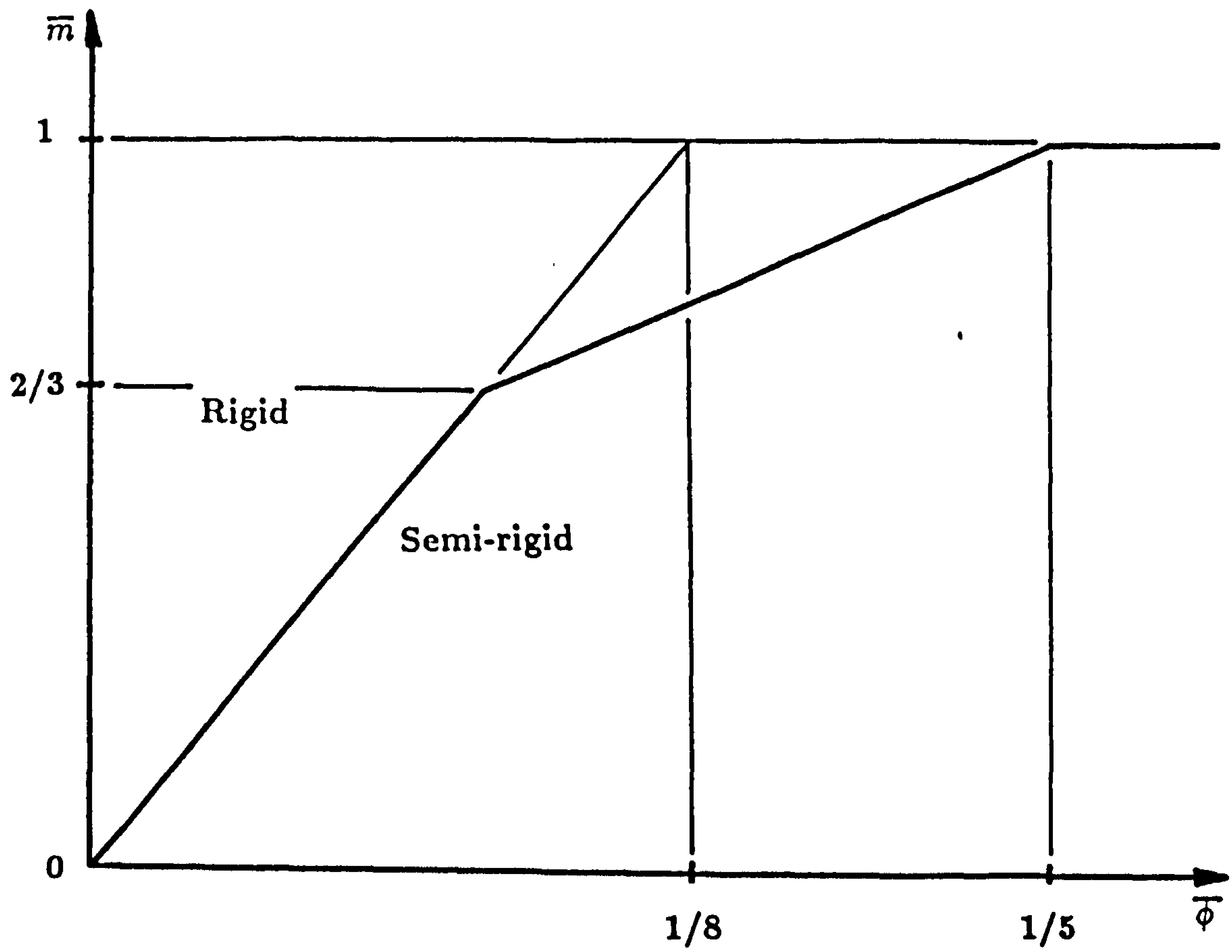


Figure 1.6 Moment-rotation characteristics.



when $\overline{m} \leq 2/3$........................$\overline{m} = 8\overline{\phi}$

when $2/3 < \overline{m} \leq 1.0$................$\overline{m} = (20\overline{\phi} + 3)/7$

with $\overline{m} = M/M_p$ and $\overline{\phi} = EI_b\phi/L_bM_p$

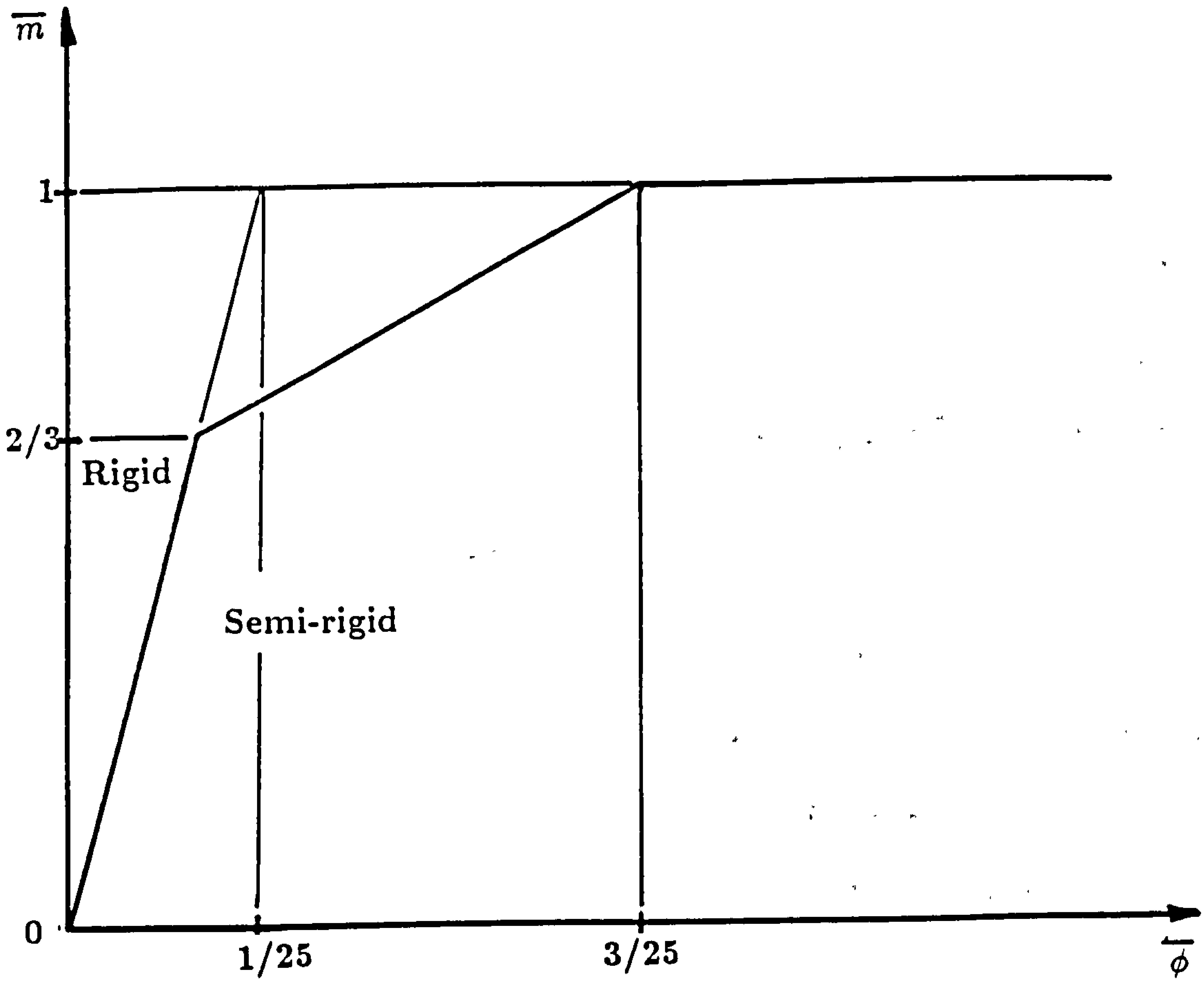
where :

M_p is plastic moment resistance of beam,

EI_b is flexural rigidity of the connected beam,and

L_b is length of the connected beam.

Figure 1.7 Classification of M-ϕ characteristics for beam-to-column connections in braced frames [1.15].



when $\overline{m} \leq 2/3$..........................$\overline{m} = 25\overline{\phi}$

when $2/3 < \overline{m} \leq 1.0$..................$\overline{m} = (25\overline{\phi} + 4)/7$

Frames in which every storey must satisfies:

$(K_b/K_c) \geq 0.1$

where:

K_b is mean value of I_b/L_b for all the beams at the top of that storey,

K_c is mean value of I_c/L_c at the columns in that storey,

L_b is span of a beam (centre-to-centre of columns),

L_c is storey height for acolumn,

I_b and I_c are second moment of area of a beam and a column respectively.

Figure 1.8 Classification of M-ϕ characteristics for beam-to-column connections in unbraced frames [1.15].

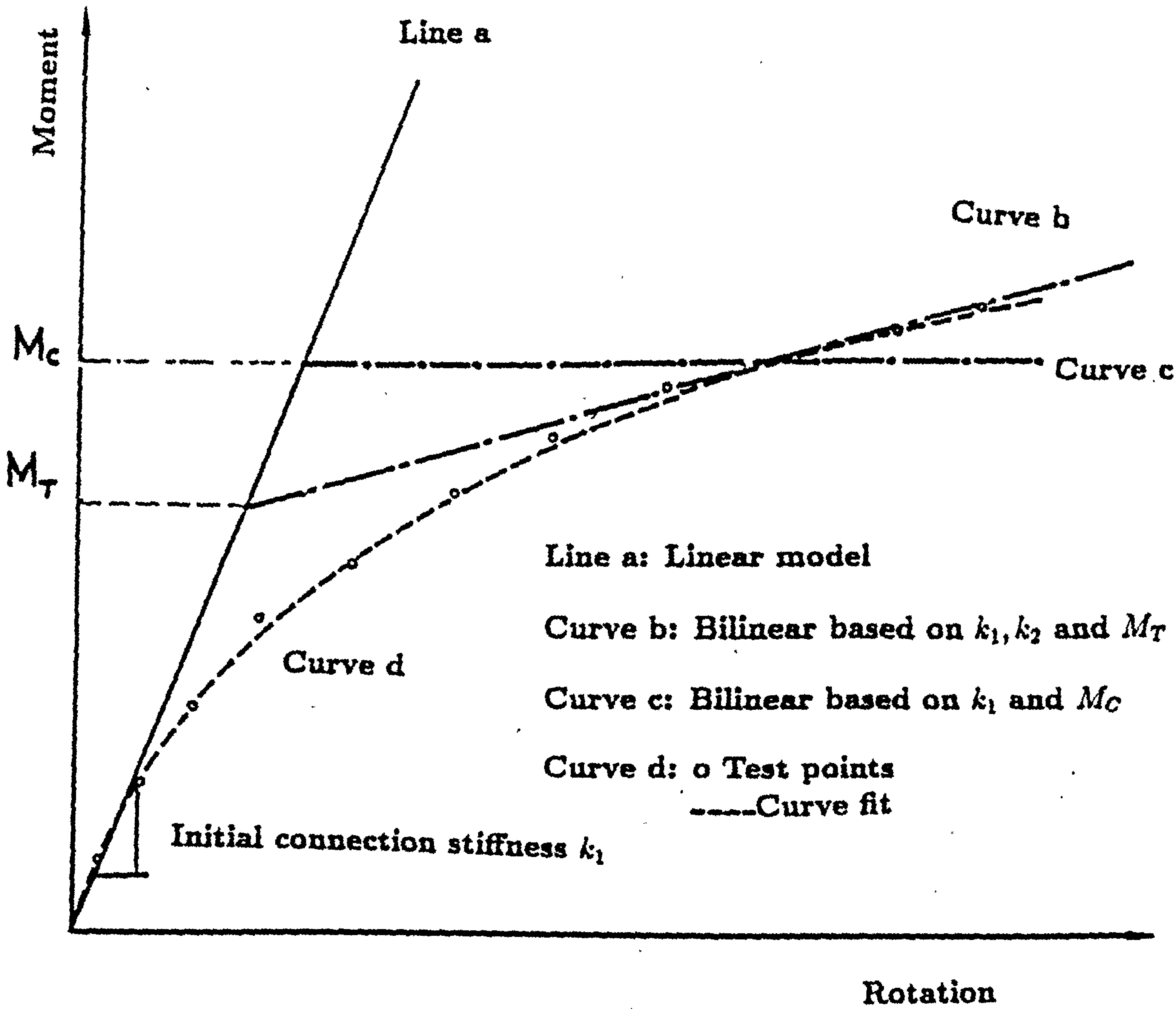


Figure 1.9 Linear and bilinear M-ϕ models.

CHAPTER II

HEADER PLATE CONNECTIONS

2.1 Introduction

A header plate connection, as shown in Figure 2.1, consists of a rectangular plate fillet welded to the end of the web of the beam. Usually the welding is to both sides of the web of the beam over the entire depth of the plate and is performed in the fabrication shop. The plate can be welded either centrally with respect to the beam's flanges or offset towards the top flange. The rectangular plate is fabricated with holes in it so that the site connection can be made by bolting the end-plate to the column flange for framing connected about the major axis, or to the web of the supporting member for framing connected about the minor axis. The end-plate is substantially smaller than the beam depth and represents about 30 to 90 per cent of the beam depth in all tests carried out so far. Pask [2.1] recommended that the end-plate depth should vary from a maximum equal to the depth between fillets of the supported beam to a minimum of approximately 0.6 of the beam depth. These connections are very popular in the construction industry [2.2] and are used mainly as "simple" connections. They are considered as shear connections with distinct appearance over the traditional double angle cleat connections. Thus, to provide satisfactory performance, the connection must transfer the end shear from the beam to the support, causing only a negligible end-moment to develop in the supported beam, and allowing virtually unrestrained rotation to occur.

The notations for the major parameters of header plate connections as shown in Figure 2.1 are identified as follows:

L_t = Distance from the tension beam flange to the top edge of the plate,

C_t = Distance from the centre of the upper fastener holes to the top edge of the plate,

P = Distance between two rows of fasteners, referred to as pitch distance,

C_c = Distance from the centre of the lower fastener holes to the bottom edge of the plate,

L_c = Distance from the compression beam flange to the bottom edge of the plate,

G = Distance between two lines of fasteners, referred as gauge distance,

L_p = Plate depth,

B_p = Plate width,

T_p = Plate thickness,

T_{wb} = Beam web thickness,

T_{fc} = Column flange thickness, and

D_b = Beam depth.

2.2 General behaviour and design recommendations

The current design practice of assuming flexible type connections to behave as pins neglects the moment in the column produced by the connections and assumes the load is transferred only through vertical shear at the end-plate face. However, experimental evidence reported by Sommer [2.3], Hafez [2.4] and Lewitt et al [2.5] have shown that these connections can offer some moment capacity of the order 5 to 25 per cent of the fixed end-moment of a beam at working loads, depending on: the type of load, the rotation of the supporting beam due to the beam connection, and the beam-span to beam depth ratio. Therefore the simplifying assumption that flexible connections resist no moment has a conservative effect on the beam design but as connections do offer a certain amount of restraint, it is beneficial to use the effect of this restraint in a further refinement of the design of the structure. For example it can be used to reduce the maximum positive moment in the beams caused by the gravity loads and to reduce the effective length factor for the column, thereby increasing its load-carrying capacity. However, the connection also transfers some moment into the column so that this member can no longer be designed as an axially loaded member.

Typical moment-rotation characteristics for such connections are shown in Figure 2.2 where two distinct phases are apparent. Experimental investigations reported the following mode of behaviour. At the initial stage, i.e. at very low loads, the connection behaves almost elastically. As the load increases, a slight opening between the top of the end plate and the column face starts to develop. This causes yielding of the upper region of the end plate and yielding of the beam web at the bottom of the connection. At this stage, the curve becomes flatter but not horizontal as the load increases. As large deformations occur from the increasing applied load, the top of the end plate starts to pull away from the face of the column at the first bolt row in tension zone. Then deformation of the end-plate continues at the top of the next lowest bolt row. The large inelastic deformation of the end-plate results in a large rotation of the connection. Increasing rotation of the connection results in the bottom flange of the beam bearing against the face of the column. As a result of this, the moment capacity of the connection increases significantly and the connection becomes much stiffer. The transition (i.e. when the lower beam flange comes into bearing against the column flange), is indicated by a sharp rise in the curve of Figure 2.2. Following the sharp increase of the moment capacity of the connection, the load later drops rapidly until failure of the end plate occurred.

Lewitt et al [2,5] reported that although differences in detailed behaviour exist, the end-plate connections behave in a similar manner to double web-angle connections except that both the initial stiffness and the maximum moment were slightly larger for the former.

Experimental observations by Zandonini and Zanon [2.7] showed that plastic deformations in the end-plate were located mainly in the vicinity of the web and longitudinally between the bolt holes. Their pattern showed the presence of significant prying action and the behaviour may be represented by a T-stub model or inelastic plate model.

In most tests using header plate connections the usual cause of ultimate collapse was observed in the end-plate at rotations considerably greater than those expected for the simply supported end of beams in steel construction. The modes of failure observed in the end-plate were: tearing of the plate material, weld pulling away from the plate, tension failure of the plate and shear failure between the weld and beam web. Bolt failures rarely occurred.

Although tests have been conducted in which joints have been taken to quite large rotations, more than 0.1 radians, studies, for both frames and individual restrained members, of the joint rotations required at maximum load have shown that behaviour at rotations beyond 0.05 radians (often much less), has little practical significance. Thus in such joint types (e.g. web cleats or header plates) for which direct bearing of the beam lower flange against the column face eventually occurs due to beam rotation, only that part of the moment-rotation curve at lower rotations is of real interest (i.e. up to the point at which the lower flange of the beam begins to bear against the face of the column). For comparison purposes between the experimental and predicted moment-rotation curve to be used in structural analysis, it is necessary to provide a limit on rotation up to which it would be desirable for the prediction equations to closely approximate connection behaviour by defining an upper bound on end rotations of flexible connections. This corresponds to the maximum rotation expected in a simply supported beam.

Therefore, it is instructive to examine the magnitude of the rotations that are likely to occur in the beam-to-column connections in a steel frame. Consider, for example, a beam of span L, pinned to supporting columns and carrying a uniformly distributed load, w. The beam end rotation is given by:

$$\theta_o = \frac{wL^3}{24EI} \tag{2.1}$$

and the maximum moment at mid-span is given by:

$$M = \frac{wL^2}{8} \tag{2.2}$$

Substutiting in equ. (2.1), gives:

$$\theta = \frac{ML}{3EI} \tag{2.3}$$

For a constant load, any restraining action resulting from semi-rigid connections would cause the end-rotation of the beam to be less than θ_o. Suppose the mid-section of the beam yields, developing the plastic resistance moment, M_p, of the section. Then the corresponding rotation θ_p is given by:

$$\theta_p = \frac{M_pL}{3EI} \tag{2.4}$$

i.e.

$$\theta_p = \frac{S\,F_y L}{3EI} \tag{2.5}$$

since S = shape factor x Z and typical shape factors for I-sections are 1.15 for bending about the x-axis and 1.6 for bending about the y-axis, therefore

$$\theta_p = 1.15\,Z\,\frac{F_y L}{3EI} \tag{2.6}$$

In the above equations, S is the plastic modulus, Z is the section modulus and F_y is the yield stress of the beam.

Thus,

$$\theta_p = 1.15\,\frac{2I}{D_b}\,\frac{F_y L}{3EI} \tag{2.7}$$

i.e.

$$\theta_p = \frac{2.30}{3E}\,F_y\left[\frac{L}{D_b}\right] \tag{2.8}$$

where D_b is the depth of the beam.

In practice, the ratio of (L/D_b) cannot be higher than 20 to 22 in beam design to avoid excessive deflections.

Therefore, for grade 43 steel, F_y = 275 N/mm², equ. (2.8) gives:

$$\theta_p \nless \frac{2.30 \times 275}{3 \times 205000} \times 22$$

$$\theta_p \nless 0.0226 \text{ radians} \tag{2.9}$$

Similarly for grade 50 steel, F = 355 N/mm², equ. (2.8) becomes:

$$\theta_p \leq \frac{2.30 \times 355}{3 \times 205000} \times 22$$

i.e.

$$\theta_p \leq 0.0292 \text{ radians} \tag{2.10}$$

Therefore, the maximum rotational deformation for a beam-to-column connection in a frame designed to meet the recommended deflection limits would be 0.0226 radians for steel grade 43. This upper bound on end rotation provides a lower limit on rotation up to which the predicted rotation should closely approximate the connection behaviour for the purpose of structural analysis.

Similarly, Packer and Morris [2.8] examined the magnitude of the rotations that are likely to occur in the beam-to-column connections in a steel frame. Based on the maximum mid span deflection and on a maximum sway limit of height of the storey over 300, the maximum rotational deformation is limited to 0.0238 radians.

In designing such connections, the objective is to provide enough rotational capacity to allow the beam to reach yield before bearing occurs. Rotational flexibility of the connection can be achieved by the use of a relatively thin end-plate assuming beam rotation will occur near the lower edge of the end-plate as shown in Figure 2.3, and with local deformation and yielding at the top row of bolts. As a "rule of thumb", the end-plate thickness should be limited to 8 mm for beams up to and including 457 x 191 UB and to 10 mm for 533 x 210 UB and over (see reference [2.1]). As a design recommendation, the length of the end-plate should be limited to a maximum of the clear depth of the web and to a minimum of 0.6 times the beam depth. The position of the upper edge of the end-plate should be near the beam upper flange. The end-plate should be continuously fillet welded to the beam web only.

For design purposes, Pask [2.1] recommended the distance between the lower edge of the end-plate and the lower beam flange, (L_1 in Figure 2.3), should be limited in order to avoid contact between beam lower flange and the column face, even at high rotation. By adopting a maximum design rotation of 0.030 radians, based on the assumption that the maximum bending moment is the yield moment, and assuming a rotation about the bottom edge of the end-plate:

$$\theta = \frac{T_p}{L_1} = 0.030 \text{ radians} \tag{2.11}$$

therefore the imposed limitation on L_1 for practical beams is:

$$\frac{L_1}{T_p} \nless 33 \tag{2.12}$$

As such end-plate connections are assumed to be subject to shear loading only and are designed for the simply supported beam end-reaction, the load capacity of each component in a connection must be checked to ensure that this factored end-reaction would not cause failure. The following items must be investigated:

i) Bolt group capacity, by checking the adequacy of the most heavily loaded bolts (BS5950 [2.9] cl. 6.3.2 and 6.3.3);
ii) local shear capacity of the beam web at the end plate; the object of this investigation is to ensure that the beam web is capable of transferring the shear force into the fillet welds (BS 5950 cl. 4.2.3);
iii) shear capacity of end-plate (BS 5950 cl. 4.2.3);
iv) weld capacity (BS 5950 cl. 6.6.5), 6 mm fillet welds are the minimum required size for use on 8 and 10 mm thick end-plates.

The behaviour of bolted beam-to-column connections can be described in terms of the factors which influence their moment-rotation characteristics. The difficult problem is to determine how the M-Φ function which is affected by variations in connection parameters such as thickness and depth of connection, beam depth, gauge distance, etc. Thus, a parametric analysis should be carried out to study the sensitivity of the M-Φ curve to the main connection parameters, where test data are available.

2.3 Parametric analysis:

The parametric analysis of the behaviour of flexible end-plate connections is based here on the available experimental test results. In general there has been a considerable amount of work done on end-plate connections over the last three decades. The majority of this work is concerned with end-plate connections used as moment connections which will be studied in the next two chapters. Shear connections have received less attention than moment end-plate connections (i.e. flush and extended end-plate).

2.3.1 Experimental data

Six series of tests on header plate connections have been identified, three conducted in Canada, two in Australia and a recent one in Italy. These are referenced in Table 2.1, with fuller details being provided in Appendix A.

The original work was carried out in Canada by Sommer [2.3] in 1969. Sommer tested twenty connections with flexible end-plates (and also four connections with header angles welded to the beam webs). The effect on the connection behaviour of end-plate thickness, beam depth, number of rows of bolts and gauges were investigated experimentally. With gauges of 4 in (101.5 mm) or 6½ in (165 mm); end-plate thickness of 1/4 in (6 mm), 3/8 in (9.5 mm) or ½ in (12.5 mm); and with

2 to 6 rows of bolts, a large variation in flexibility was achieved. All bolt holes were 3/4 in (19 mm) diameter A325 bolts. The connections were subjected to combined shear and moment and the resulting moment-rotation curves were obtained. In addition, the following conclusions were drawn:

i) the behaviour of header plate connections depends on the length of plate, the thickness of the end-plate, the beam web thickness, the connection gauge distance and the number of bolt rows;

ii) end-plate connections display moment-rotation characteristics similar to those of angle connections. Differences in behaviour between header plate and web angle connections may be attributed to differences in geometry of the two connections;

iii) end-plate connections can provide adequate shear connections when designed for shear only;

iv) end-plate connections can be designed for a wide range of flexibilities;

v) in deep connections the possibility of bolt fracture should be investigated;

vi) header plates have adequate ductility to accommodate the end rotations developed in a typical simply supported beam carrying a uniformly distributed load, sufficiently large to cause yielding of the cross section of mid-span, provided the length-to-depth ratio of the beam does not exceed 24;

vii) using the yield beam line concept (discussed in section 5.2.1 of Chapter 5), the tests showed that the header plate connections developed from 4% to 25% of the yield moment at the yield beam line and that the corresponding rotation was about 90% of the free end rotation of the associated beam. It was concluded that flexible end-plate connections were quite adequate as simple shear connections and that the bolts and welds associated with the connections should be designed to transmit shear only.

Moment-rotation curves for the 20 tests are available in references [2.3, 2.6, 2.10 and 2.11].

Bennetts et al [2.12] conducted four tests on shear connections for beam-to-column joints, with the primary load on the connection being shear but with the beam end also rotating. Two specimens employed double angle cleats and two used flexible end plate connections. The testing was done in a cantilever mode with a column stub of 250 UC 105. A concentrated load was applied to the beam in a position designed to produce high shear on the connection, significant rotation of the end of the beam while preventing premature beam bending failure. The interface moment, the relative beam column rotation between beam and column, end shear and slip between the connection and the beam were all measured. The tests showed that the header plate connections developed only 3 to 5% of the fixed end moment at the failure point. M-Φ curves are provided for each test in reference [2.12].

Phum and Mansell [2.13, 2.14] conducted an experimental test programme on shear connections. The twelve connections tested included seven flexible end-plate connections with varying beam depth, end-plate depth and number of bolt rows. The specimens were tested with monotonically increasing deformation until the maximum load. All bolts were M20, in 22 mm diameter holes and were torqued to be snug tight. There was some retesting of four specimens (F_2, F_3, F_4 and F_6) because the high capacity of the connections causing failure to occur first in the beam. For the second test on each specimen the span was shortened to increase the ratio of shear to bending moment, increasing the likelihood of a connection failure occurring before flexural failure of the beam. The variations between the duplicate tests included both the variability between specimens and variability due to experimental procedure. The rigs used were not identical. The test results showed that the non-linearity originated in both the connection and the beam. The capacity of the connection is controlled by the ability of the web to transmit shear into the end-plate.

The applied shear force versus the vertical deflection and the connection's rotation were recorded for each specimen. It is not possible to convert the curves presented by Phum and Mansell [2.13, 2.14] into moment-rotation curves since the precise location of the gauges used to measure vertical deflection, as well as the exact location of the point load, are not known explicitly.

In 1982, Hafez [2.4] investigated both analytically and experimentally flexible end-plate connections. The testing was done in a cantilever mode with a comparatively large lever arm. Eight static tests on header plates of varying geometry such as end-plate thickness, horizontal distance between bolt holes (gauge distance), number of bolt rows and overall depth of connection were conducted by subjecting the specimens to shear and moment. Only one beam size was used, a WWF 27 x 106 equivalent to WWF 690 x 158. Hafez showed that the flexible end-plate possesses similar characteristics to comparable double angle cleats and all the parameters varied in the tests influenced the stiffness of the connection. The rotation of the beam relative to the column was measured, the moment-rotation relationship being found to be non-linear. Moment-rotation data is provided in references [2.4, 2.15] for the eight test specimens.

In 1986, Zandonini and Zanon [2.7] conducted an extensive experimental investigation on semi-rigid connections. In total 19 specimens were selected in order to investigate a wide range of stiffness and moment capacity. The flush and extended plate tests will be reviewed in Chapter 3 and 4 respectively. The flexible connections investigated were double web cleats and header plate. Only three header plate connections were considered, with two, three and four bolt rows for each end-plate, while the plate thickness was kept constant to 12 mm. Bolts grade 4.8 with 20 mm in diameter were used in the first two arrangements, while bolts grade 6.8 with 16 mm in diameter were used in the connection with four bolt rows. The bolts were preloaded to about 40% of their yield strength in order to simulate the snug tight condition.

The loading process was conceived as a series of loading cycles, consisting of a loading branch followed by a complete unloading (no load reversal). The envelope curve of the loading branch is given in Appendix A. On the basis of this test programme, the authors concluded that:

i) the moment capacity ranged between the 17% and 31% of the beam's plastic moment;
ii) the contribution of the bolt elongation to the overall rotation was of about 15 to 20% for the stronger connections and 55% in test HPl-1;
iii) the number of bolt rows influenced the moment capacity of the connection, the initial and the strain hardening stiffness;
iv) the elastic stiffness was only modestly affected by the bolt preloading.

Moment-rotation curves for the tested specimens are available in reference [2.7].

Recently in 1988 MacIntyre [2.16] conducted an experimental investigation on shear end-plate connections. In total 12 test specimens were carried out, 9 were clipped end-plate which are not relevant to this study, while three were conventional shear end-plates, referred to as unclipped end-plates by the author. The unclipped end-plate specimens were made as similar as possible to those conducted by Sommer [2.3], to establish if any differences subsequently found between the two investigations could be attributed to intentional differences in controlled variables, rather than to differences in the fabrication of the specimens, the design of the test rig or the overall test procedure.

The test frame consisted of a vertical column stub W360 x 64 connected to a horizontal beam, with a brace joined to each to allow the column to resist moment. The load was positioned over the test beam, 1.2 m from the face of the column stub. The main variable connection parameters were the number of rows of bolts, plate thickness, gauge distance and beam depth. The relative rotation between the test beam axis and the column axis was measured and plots of the moment-rotation curves for each of the 12 tests are presented in references [2.16, 2.17].

Some of the major observations through the test series were that non linearity was observed early in the loading sequence. The moment carried by the connection at the point when the bottom flange of the beam began to bear against the column, M_b, was a small proportion of the plastic moment of the beam section, M_p, varying from 0.07 M_p to 0.16 M_p. The large range reflects the influence of variations in the number of rows of bolts and beam depth. The ultimate moments developed by the connections were well in excess of M_b because of the increased lever arm of the connection as the line of action of the resultant compressive force moved from the lower region of the end-plate to the bottom flange of the beam.

The aim of all the tests was to investigate the effect on the connection behaviour of end-plate thickness, beam depth, depth of the end-plate, number of bolts and gauge of fasteners to the column face. None of the available test series investigated the effect of material yield stress and column properties.

It is important to note that the M-Φ curves obtained by Bennetts et al [2.12] showed that after the moment reached a peak value, the rotation increased further but the moment dropped significantly (see Figure 2.4). However the moment-rotation curves obtained in the other investigations were such that the rotation increased as the applied moment was increased. A possible explanation of this apparent inconsistency is that the falling branch is developed due to lack of ability to rotate at constant moment.

2.3.2 Analysis of moment-rotation characteristics

The behaviour of bolted beam-to-column end-plate connections can be described in terms of the factors which influence the moment-rotation characteristics. The difficult problem is to determine how the M-Φ function is affected by variations in connection parameters such as thickness and depth of connection, beam depth, gauge distance, etc. Comparative moment-rotation curves are used to describe and illustrate the effect of the main parameters on the overall connection performance, in order to permit the construction of M-Φ functions for connection geometries other than those that have been tested. The comparison of the M-Φ curves is made for the portion of the curves up to the point at which the beam flange bears against the column face and for a series of connections for which the significant geometric parameters are varied one at a time.

2.3.2.1 *Effect of end-plate thickness*

Figure 2.5a shows the effect of increasing thickness of the connection plate over the range (6.359, 9.525 and 12.700 mm) of Sommer's [2.3] results. As the thickness is increased, the moment capacity is increased and the rotation at which the beam flange hits the column is also greater because of the increased distance of the beam flange from the column face. However in Figure 2.5b where the M-Φ curves represent the same range of end-plate thickness as in Figure 2.5a but with a shallower beam and consequently smaller depth of end-plate and smaller number of bolts, the effect of the end-plate thickness is less apparent. The moment at which bearing of the lower beam flange against the column flange begins is smaller for thin end-plate. Figures 2.5c and 2.5d clearly indicate that the end-plate thickness has a strong influence parameter on the moment-rotation curves. In Sommer's test

12 of Figure 2.5c and in Hafez's test 8 of Figure 2.5d, the connections failed before the beam hit the support. In all cases, a thicker end-plate corresponds to a stiffer connection.

2.3.2.2 *Effect of connection depth*

Figures 2.6a and 2.6b show families of moment-rotation curves where the only variable in each test is the depth of the end-plate connection. It is clear from these plots that increasing the connection depth leads to an increase in connection stiffness, in moment capacity of the connection and usually in the rotation values at which the lower beam flange comes into bearing against the column. It should be noticed that increasing the depth of the connection leads to an increase in the number of bolt rows and consequently a decrease in pitch distance (i.e. vertical distance between bolt centres).

2.3.2.3 *Effect of beam depth*

The M-Φ curves for tests 6, 26 and 27 of Sommer corresponding respectively to a beam section of W24 x 76, W18 x 45 and W12 x 27 in Figure 2.7a show the effect of increasing the depth of the beam on the stiffness of the connection. Figure 2.7b shows the variation of M-Φ curves for Sommer's tests 10 and 13. In both figures, the increase of beam depth has negligible effect on M-Φ curve shape before the beam's bottom flange hits the column face. However, after bearing occurs, beam depth has a significant effect on the connection stiffness. In this later stage, the connections attract higher moment as the beam depth increases.

2.3.2.4 *Effect of gauge distance*

In order to study the effect of the bolt cross centre distance on connection behaviour, M-Φ curves for Sommer's tests 8 and 18 are plotted in Figures 2.8a and for Hafez's tests 1 and 3 in Figure 2.8b. It can be seen that in both figures, increase of gauge distance from 101.60 mm to 139.70 mm causes a decrease in the connection's stiffness and in the moment at which bearing begins. It has been pointed out by many investigators that the bolts should be placed as close as possible to the web of the beam to obtain the maximum connection stiffness.

2.3.2.5 *Effect of pitch distance*

The tests where the vertical distance between the centres of two row bolts is variable are tests HPl-1 and HPl-2 carried out by Zandonini and Zanon [2.7]. These are shown in Figure 2.9 in which only the monotonic increasing moment is depicted on the M-Φ curves (i.e. no unloading). Figure 2.9 indicates that a decrease in pitch distance from 120 mm to 60 mm with an extra bolt row leads to an increase in connection stiffness and in moment capacity of the connection.

Investigation of the effects of weld size or form of weld, column proprieties and physical properties of the material was omitted from the above comparison of the M-Φ curves because of lack of experimental data dealing with such variables. As a result of this parametric analysis, with reference to Figure 2.1 the most significant parameters necessary to establish the relationship between moment and rotation of a flexible end-plate connections are:

i) the end-plate thickness, T_p

ii) the gauge distance, G

iii) the pitch distance, P

iv) the depth of the end-plate, L_p

v) the depth of the beam, D_b

vi) the type and number of bolts which is related to bolt pitch distance.

However, the initial tension on a bolt loaded in shear does not affect the ultimate shear capacity, so that this discrepancy is not significant [2.18]. According to

Zandonini and Zanon [2.7], the elastic stiffness without bolt preloading was only modestly affected by this parameter.

As mentioned in Section 1.5 of Chapter 1, knowledge of the M-Φ data either from experimental results or from analytical work is a prerequisite for performing any sort of frame analysis with semi-rigid connections. However, the M-Φ curve based on experimental results cannot be available for all the connections required because of the variations in configuration and therefore each joint has its own characteristic M-Φ curve. Thus, an analytical representation of the moment-rotation curve is necessary to overcome this problem.

2.4 Analytical representation of moment-rotation curves

The review of the available methods for the prediction of beam-to-column M-Φ behaviour are discussed in Section 1.5. These models are generally either an approximation based on curve fitting to test data, or behavioural and mechanical models, or a sophisticated numerical simulation. These prediction equations, already described in Section 1.5, for the moment-rotation relationships are very effective for design and analysis purposes.

For a header plate connection, if bearing of the beam bottom flange against the column face occurs due to beam rotation, large connection rotation may subsequently occur. In predicting the moment-rotation curve, it is not necessary to consider the rotation beyond this bearing point because it is the part of the M-Φ curve at lower rotation that is of real interest [2.19] for both frames and individual restrained members. Therefore, the prediction equation of moment-rotation relationship is applicable only to the point at which the bottom flange of the beam bears against the column face.

2.4.1 Sommer's model

Sommer [2.3] used results from the tests he carried out to represent the moment-rotation curve in the form of a power series relating the rotation, Φ, to the scaled moment KM. The standardisation constant, K, for a connection depends on the connection size parameters and has the form:

$$K = \prod_{j=1}^{m} q_j^{a_j} \tag{2.13}$$

where

q_j = numerical value of jth size parameter,

a_j = a dimensionless exponent which indicates the effect of jth size parameter on the M-Φ relationship, and

m = total number of size parameters.

The exponents a_j were determined by considering a family of experimental moment-rotation curves for connections differing only in parameter p_j as shown in Figure 2.10. At a particular rotation, each pair of curves were assumed to have the form:

$$M_1/M_2 = [q_{j_2}/q_{j_1}]^{a_j} \tag{2.14}$$

where q_{j_1} and q_{j_2} are the numerical values of parameter q_j for connections 1 and 2 respectively. Rewriting equ. (2.14)

$$a_j = [\text{Log}\ (M_1/M_2)]\ /\ [\text{Log}\ (q_{j_2}/q_{j_1})] \tag{2.15}$$

The exponents a_j were calculated using equ. (2.15), for several rotations, for every available combination of experimental curves, such as 1 and 2, 1 and 3, and 2 and

3, etc. The mean value was then adopted from all pairs of curves for connections which differed in the value of only one parameter.

Sommer standardised the results of his tests on flexible end-plate connections in this fashion. The final values determined for the exponent, a, were as follows:

for plate thickness T_p, $a = -1.6$

for gauge distance G, $a = +1.6$

for connection depth L_p, $a = -2.3$

for beam web thickness T_{wb}, $a = -0.5$

The effect of all the parameters was combined to give the standardisation constant "K" recommended by Sommer where,

$$K = T_p^{-1.6} \, . \, G^{+1.6} \, . \, L_p^{-2.3} \, . \, T_{wb}^{-0.5} \tag{2.16}$$

when average values were calculated for all m exponents a_j in equ. (2.13), they were plotted on a standardised moment-rotation curve (KM versus Φ) and a least squares curve fitting procedure then used to derive the final prediction equation. This is as follows,

$$\Phi = 5.1 \times 10^{-5} \,(KM) + 6.2 \times 10^{-10} \,(KM)^3 + 2.4 \times 10^{-13} \,(KM)^5 \tag{2.17}$$

where K the standardisation constant is given by equ. (2.16), and the connection parameters are shown in Figure 2.1. In the predicted M-Φ relation, the moment, M, is in kips-inches and the connection parameters in inches. The rotation, Φ, is in radians.

Frye and Morris [2.20] extended the work developed by Sommer [2.3] to six other type of beam-to-column connections. In reference [2.20], Frye and Morris claimed that equations (2.16) and (2.17) are in dimensionless form as well as prediction equations for all the other six types of connections.

In metric units, the prediction equation is given by:

$$\Phi = 6.14 \times 10^{-5}(KM) + 1.08 \times 10^{-9}(KM)^3 + 6.06 \times 10^{-13}(KM)^5 \qquad (2.18)$$

and

$$K = T_p^{-1.6} \cdot G^{+1.6} \cdot L_p^{-2.3} \cdot T_{wb}^{-0.5} \qquad (2.19)$$

where in equations (2.18) and (2.19),

Φ is rotation of the connection in radians,

M is moment of the connection in kNcm, and

K is standardisation constant in $cm^{-2.8}$.

The predicted curve is applicable only to the point when first contact is made by the lower flange with the column face and a maximum deviation of 4% of the standardised curve from experimental curve was claimed by Frye and Morris [2.20].

2.4.2 Ang and Morris's model

As an alternative to the polynomial of equation (2.18) proposed by Sommer [2.3], Ang and Morris [2.21] derived an exponential function based on Sommer's test results to express the standardised moment-rotation behaviour in the following form:

$$\frac{\Phi}{\Phi_0} = \frac{KM}{(KM)_0}\left[1 + \left(\frac{KM}{(KM)_0}\right)^{n-1}\right] \qquad (2.20)$$

where Φ_0, $(KM)_0$ and n are constants that define the shape of the standardised function. As defined in Figure 2.11, $2\Phi_0$ and $(KM)_0$ are constants through which a family of Ramberg-Osgood [2.22] curves passes. The technique used to evaluate

K is the same as defined by Sommer [2.3]. The resulting equation is:

$$\frac{\Phi}{7.04\times10^{-3}} = \frac{KM}{186.77}\left[1 + \left(\frac{KM}{186.77}\right)^{3.32}\right] \tag{2.21}$$

where

$$K = T_p^{-1.54} \;.\; G^{2.12} \;.\; L_p^{-2.41} \; T_{wb}^{-0.45} \tag{2.22}$$

The parameters T_p, G, L_p and T_{wb} are defined in Figure 2.1 and are in inches. In equ. (2.21), Φ and M are the rotational deformation of the connection in radians and the moment in kips-inches resisted by it.

In metric units, equations (2.21) and (2.22) become:

$$\frac{\Phi}{7.04\times10^{-3}} = \frac{KM}{17674.03}\left[1 + \left(\frac{KM}{17674.03}\right)^{3.32}\right] \tag{2.23}$$

and

$$K = T_p^{-1.54} \;.\; G^{2.12} \;.\; L_p^{-2.41} \;.\; T_{wb}^{-0.45} \tag{2.24}$$

where in equations (2.23) and (2.24),

Φ is rotation of the connection in radians,

M is moment of the connection in kNcm, and

K is standardised constant in $cm^{-2.28}$.

According to Ang and Morris [2.21], the maximum deviation of the calculated connection moment from the experimentally measured value is 12%, occurring for only one connection; in all other cases, the deviation was within 5%.

2.4.3 Kriviak and Kennedy's model

Hafez [2.4] concluded that the location of the neutral axis of the connection, tensile force distribution and the compressive force distribution (from deformation of all elements above and below the neutral axis) and end-plate deformation relationships, are considered to be the most significant factors necessary to establish the relationship between moment and rotation of flexible end-plate connections. He also observed from the eight tests he conducted on header-plate connections that the deformation of the connection varied linearly over its depth and that the connection rotation increased as the location of the neutral axis moved downwards. As the stiffness of the compression region differs from that of the tension zone, he conducted extensive tension and compression tests on tee stub as shown in Figure 2.12. In this model, the stem of T-section simulates the web of the beam and the flange simulates the end-plate. In the 28 tests carried out, plastic hinges formed in the end-plate along the toes of the welds and along a line approximately along the inside edges of the bolt holes.

Based on the analysis of the test results of T-sections in tension and compression, a computer program was developed to calculate the end-moment for a given rotation. A trial and error location procedure for the position of the neutral axis was used. The portion on the connection on either side of the axis is divided into a number of elements over the depth of the connection. The neutral axis is about mid-height for small rotations and moved downwards as the rotation increases. With the deformation of each element known, the forces developed in the elements based on the tension and compression load-deformation relationships are determined, as shown in Figure 2.13. These elemental forces are summed and the position of the neutral axis is modified until the sum is zero. The sum of the moments of the elemental forces for this position gives the moment in the connection for the assumed rotation [2.23].

The original Hafez load-defomation relationships for tension and compression zones are given in references [2.4 and 2.15]. The modified relationship by Hafez and Kennedy [2.15] and more recently by Kriviak and Kennedy [2.24] are given in detail in references [2.23 and 2.24], along with the assumptions made.

The computer program developed to generate the moment rotation curves is listed in reference [2.23]. The model takes into account the yield and ultimate strengths for the end-plate and beam web, the thickness of the end-plate, gauge distance, depth of connection and thickness of web plate. The complete analytical process is concerned only with the M-Φ relationships prior to the beam flange coming into contact with the column face. The analytical model also gives the moment and rotation at the occurrence of bottom flange bearing. Comparisons against both Sommer's [2.3] and Hafez's [2.4] tests showed the ratio of the test-to-predicted moment at bearing varied from 0.784 to 1.230, with the mean value of 1.06 and a relatively small coefficient of variation of 0.110. The ratio of test-to-predicted rotation at bearing varied from 0.495 to 1.90 with a mean value of 1.08 and a large coefficient in variation of 0.273 as given in reference [2.24]. Hafez and Kennedy [2.15] recommended that the rotation at contact should be taken as two-thirds of the predicted value to allow for the variation of analytical results from the experimental values. None of these investigators has stated the maximum deviation of the connection rotation calculated by using the analytical model from the experimentally measured rotation.

2.5 Comparison of experimental and analytical M-Φ curves

2.5.1 Sommer's prediction equation

The polynominal function given by equations (2.18) and (2.19) derived by Sommer [2.3] is based on a statistical analysis of his own experimental data which covers a wide range of connection parameters. The analytical model does not take into account bolt elongation and preloading, the actual yield strength and the ultimate strength of beam web and end-plate material. In Figures 2.14 through to 2.47 the analytical relationships developed by Sommer [2.3], denoted by Frye and Morris model, and Ang and Morris [2.21] are compared with all the collected test results given in Table 2.1. Figures 2.14 to 2.33 show comparison between predicted M-Φ curves against the data used by the investigator [2.3] to develop his prediction equation (i.e. equ. (2.17)). Certainly as shown in the figures, the prediction equation (2.17) predicts extremely well, within 4% as claimed by the investigators, the connection behaviour up to rotation value corresponding to bottom beam flange comes into bearing against the column face. Examinations of these plots indicates that the initial tangent stiffness is extremely well predicted. The polynomial equation showed no reverse curvature and no negative slope for all the range of values checked. The M-Φ curves predicted by Sommer seems to approximate connection behaviour with reasonable accuracy up to rotation of about 0.025 radians. They underestimate the connection stiffness in most cases except for thicker end-plate connection tests (such as Sommer's tests 19 and 20; Hafez's tests 3, 5, 6 and 7 and in all the three tests carried out by Zandonini and Zanon). In general the prediction equation underestimates the connection tangent stiffness, in most cases in the range of 10 to 15% for a rotation of less about 0.025 radians and not by 4% as claimed by Frye and Morris [2.20]. Tests performed by other investigators than Sommer (such as Hafez, Zandonini and Zanon and MacIntyre) which were not included in developing Sommer's analytical model are shown in Figures 2.34 to 2.47. These show connection behaviour to be fairly consistent. The initial stiffness of the connection for tests shown in Figures 2.34, 2.36, 2.39, 2.40 and 2.45 are extremely well predicted. However there is a slight discrepancy between prediction and experimental curves after the initial loading. The present author could not find any apparent reason for such discrepancy. Figures 2.48 to 2.50 show a comparison of moment-rotation curves for header plate connections from MacIntyre [2.16] with moment-rotation curves of similar connections tested by Sommer [2.3], designated by the letter S, over the entire range of the curve. In Figure 2.48, bearing begins at approximately 0.036 radians in connection S27 and at 0.091 radians in connection U-3-10-101-310. As discussed in section 2.3.2.3, the rotation when bearing begins is sensitive to the gap between the bottom flange of the beam and the face of the column. The size of this gap depends on the actual plate thickness and on the gap between the end of the web of the beam and the plate after welding. Connection U-3-10-101-310 has a plate thickness of 10 mm while connection S27 has a plate thickness of 6.3 mm. Before bearing occurs and also at failure, the moment carried by U-3-10-101-310 is greater than the one carried by S27, as might be expected for a thicker plate with a greater resistance to bending deformation. For example, at a rotation of 0.010 radians the moments carried by connection U-3-10-101-310 is 23 kNm and 10kNm for S27 connection. Thus, at this rotation, the ratio of these two moments is about 2.3. Using the prediction equations (2.18) and (2.19) derived by Sommer, the ratio of the two moments should be given by the factor $T_p^{-1.6}$ which is $(6.3/10)^{-1.6}$ or 2.1. Therefore, according to Sommer's prediction equation, the moment carried by connection U-3-10-101-310 is 2.1 times that for connection S27. This is in a close agreement with the test results. The curves of Figures 2.49 and 2.50 also represents connections with the same configuration. The curves are in good agreement with the exception of the point on the curves when bearing occurs and for the ultimate moment. This discrepancy is attributed to the weld size as the failure occurred in the region of the fillet weld of the end-plate to the beam web.

Comparison of the experimental curves with the standardised moment-rotation curve proposed by Sommer shows that the test results agree quite favourably with the analytical moment-rotation relationships before bearing begins.

2.5.2 Ang and Morris's prediction equation

The exponential function given by equations (2.20) and (2.21) derived by Ang and Morris [2.21] is based on similar technique as Sommer model and on Sommer's test results. Therefore, as mentioned in Section 2.5.1 the model does not take into account of bolt's elongation, bolt's preloading, weld size and of the yield stress of the material.

The M-Φ curves predicted by Ang and Morris [2.21], shown in Figures 2.14 to 2.47 for all the collected data, give relationships which are fairly close to those of Sommer [2.3] but with an over estimation of connection stiffness in most cases. This over estimation of the connection stiffness is about 10 to 20% and not by 5% as claimed by Ang and Morris [2.21].

Both analytical models, Sommer and Ang & Morris, do not predict bearing of the bottom flange of the beam against the support which is of limited significance for analysis of a typical steel framed structures.

2.5.3 **Kriviak and Kennedy's prediction equation**

A comparison of both Sommer's [2.3] and Hafez's [2.4] tests against the analytical moment-rotation curves developed by Kriviak and Kennedy [2.24], Kennedy and Hafez [2.15] and Hafez [2.4] are given by Kriviak and Kennedy in reference [2.23]. In all 28 tests at relatively small rotations, less about 0.01 radians, the analytical curves lie above the experimental ones and this results in an over estimation of the initial tangent stiffness by more than 80%. As reported by Hafez [2.4] this is due to the fact that the tension behaviour has been modelled as elasto-plastic without allowing for gradual yielding. In general, for tests with a deep beam section (W24 x 76) such as used in tests 6 to 9 and in tests 15 to 20 carried out by Sommer [2.3], the analysis over estimated the moment by about 70%. For tests with shallow beam section such as W18 x 45 or W12 x 27, the analysis predicts reasonably well the connection rotations. Again as stated in Section 2.4.3, the model provides a reasonable agreement for moment but rather more erratic prediction for the corresponding rotation when bearing of the bottom flange of the beam against the support occurs.

The analytical model does not take into account of bolt elongation, the relatively small deformation in the welds and the deformation in the beam web.

2.6 Conclusions

Header plate connections are very flexible and their effect of their end-restraint has been neglected in most designs. These connections can induce moments ranging from 5% to 25% of the yield moment capacity of the beam. This can be utilised to reduce the beam size and the effective length of the column. Header plate connections may optionally be treated as semi-rigid connections (see Section 1.4). To be conservative in beam design, an under estimation of end moment is preferred. However, over-prediction of stiffness might be preferable for safe column design because of higher moment but on the other hand the effective length goes down.

The stiffness predicted by Kennedy and Kriviak deviates from the secant of the M-Φ curve and is in considerable error in the working range of the connections which would be between 0.01 and 0.02 radians.

Both models, Ang & Morris and Sommer take into account the same connection parameters (i.e. T_p, G, L_p and T_{wb}) in predicting the moment-rotation curve. They provide a relatistic representation of connection behaviour up to rotation values of about 0.025 radians for end-plate thickness less than 12 mm. There is a very little difference between the two predicted curves.

To achieve a close approximation of the theoretical bearing moment and ultimate moment capacity of the connection, the model should be based on the failure of some component of the connection and not just on geometrical considerations. The ultimate moment capacity of a shear end-plate connection appears to be of little relevance in practice as the rotation associated with that moment is well beyond the end-rotation which can be achieved in beams used in typical structures.

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Table 2.1 Available Experimental Data for Header Plate Connections

Reference of experimental data	Date	Country of origin	Number of tests	Type of fastener	Test type	Comments on M-Φ curves
Sommer [2.3]	1969	Canada	20	¾" A325 bolts	Cantilever	M-Φ curves available for each test
Bennetts et al [2.12]	1978	Australia	2	M20 Gr 8.8 bolts	Propped cantilever	M-Φ curves available for each test
Pham & Mansell [2.13, 2.14]	1982	Australia	7	M20 Gr 8.8 bolts	Propped cantilever	Only shear - Φ, shear deflection curves provided
Hafez [2.4]	1982	Canada	8	¾" A325 bolts	Cantilever	M-Φ curves available for each test
Zandonini & Zanon [2.7]	1986	Italy	3	M20 Gr 4.8 bolts and M16 Gr 6.8 bolts	Cantilever	M-Φ curves available for for each test
MacIntyre [2.16]	1988	Canada	3	¾in. A325 bolts	Cantilever	M-Φ curves available for each test

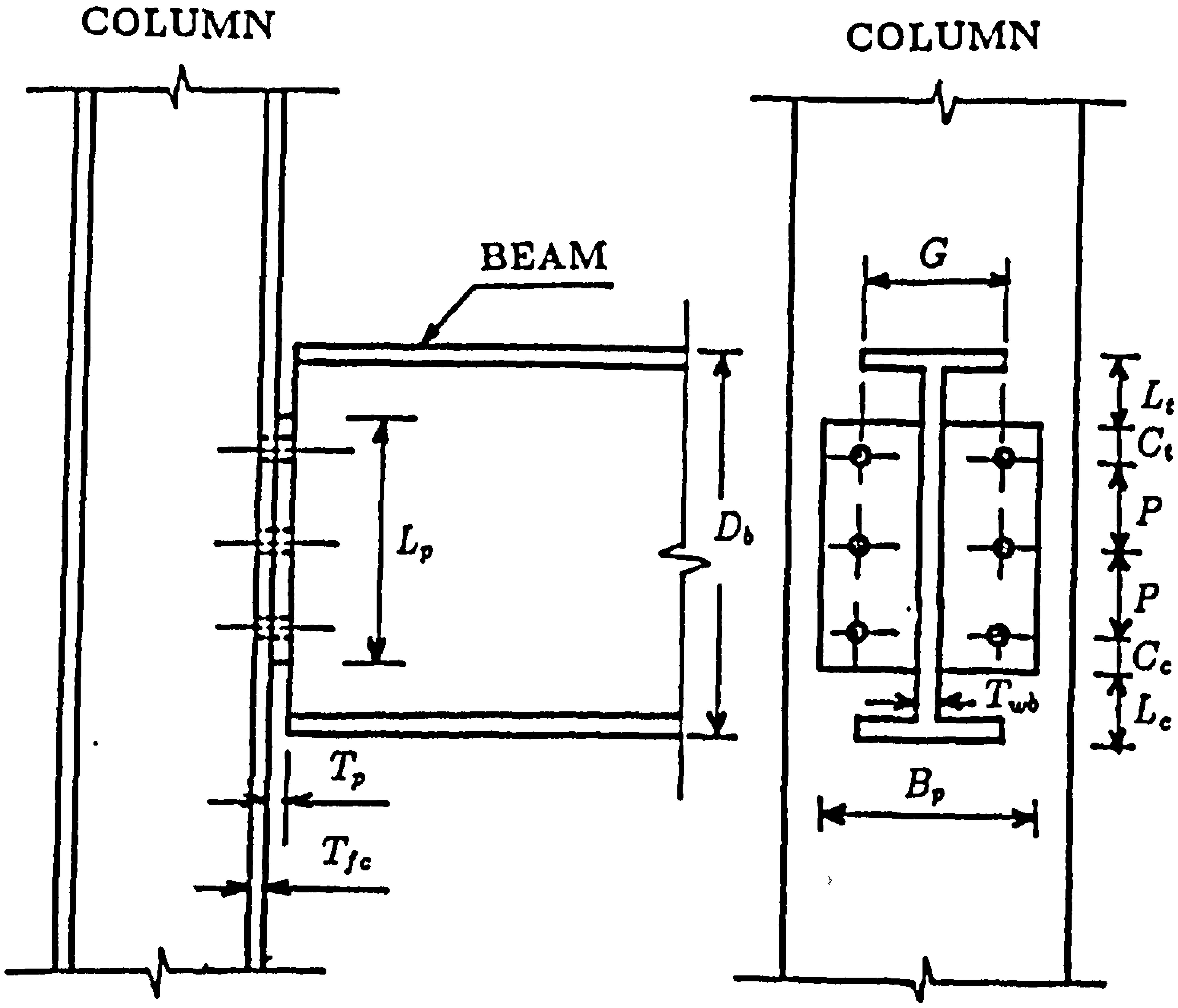
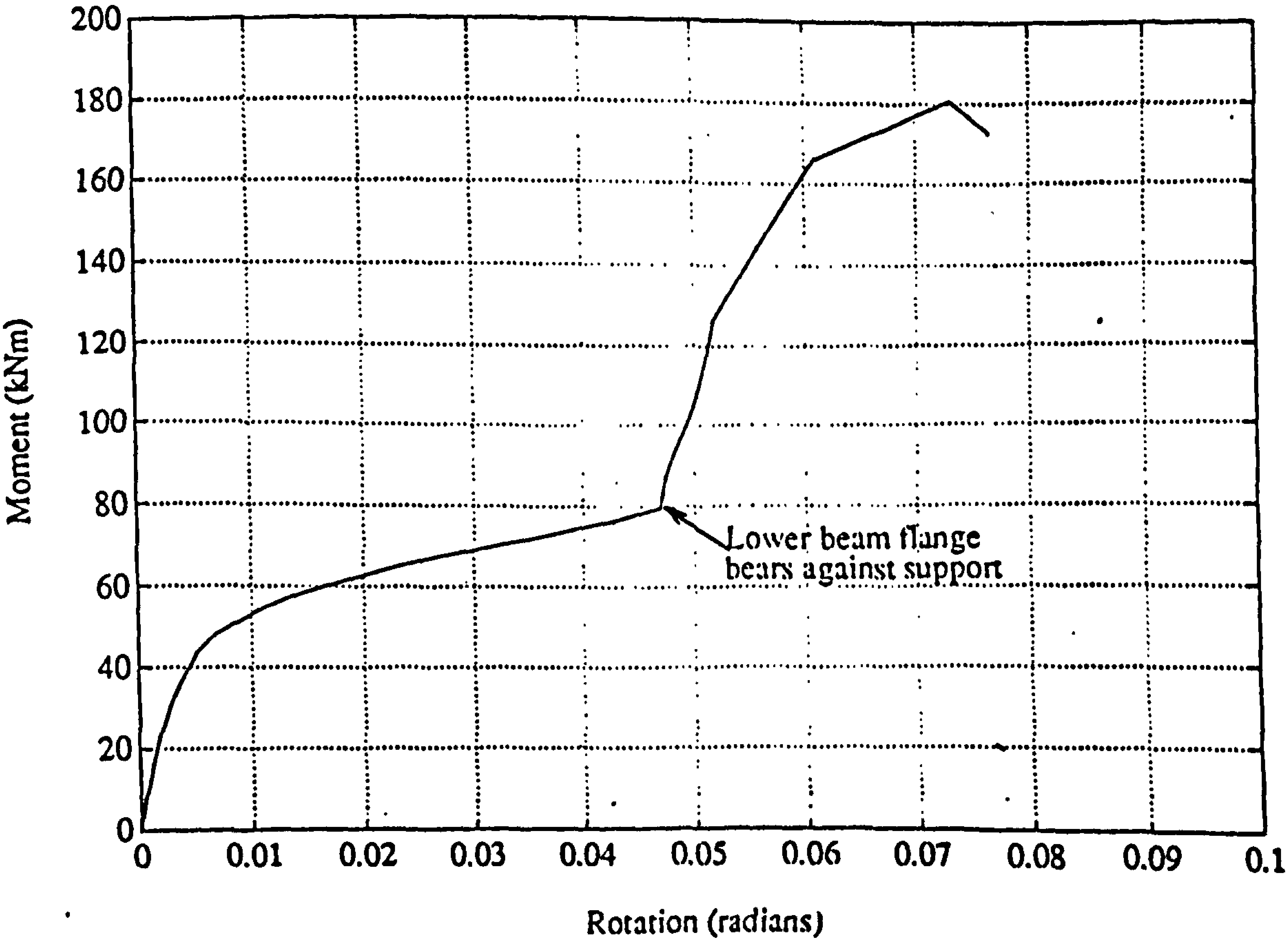


Figure 2.1 Typical header plate connection.

FIG.2-2 Moment-rotation curve for header plate connection



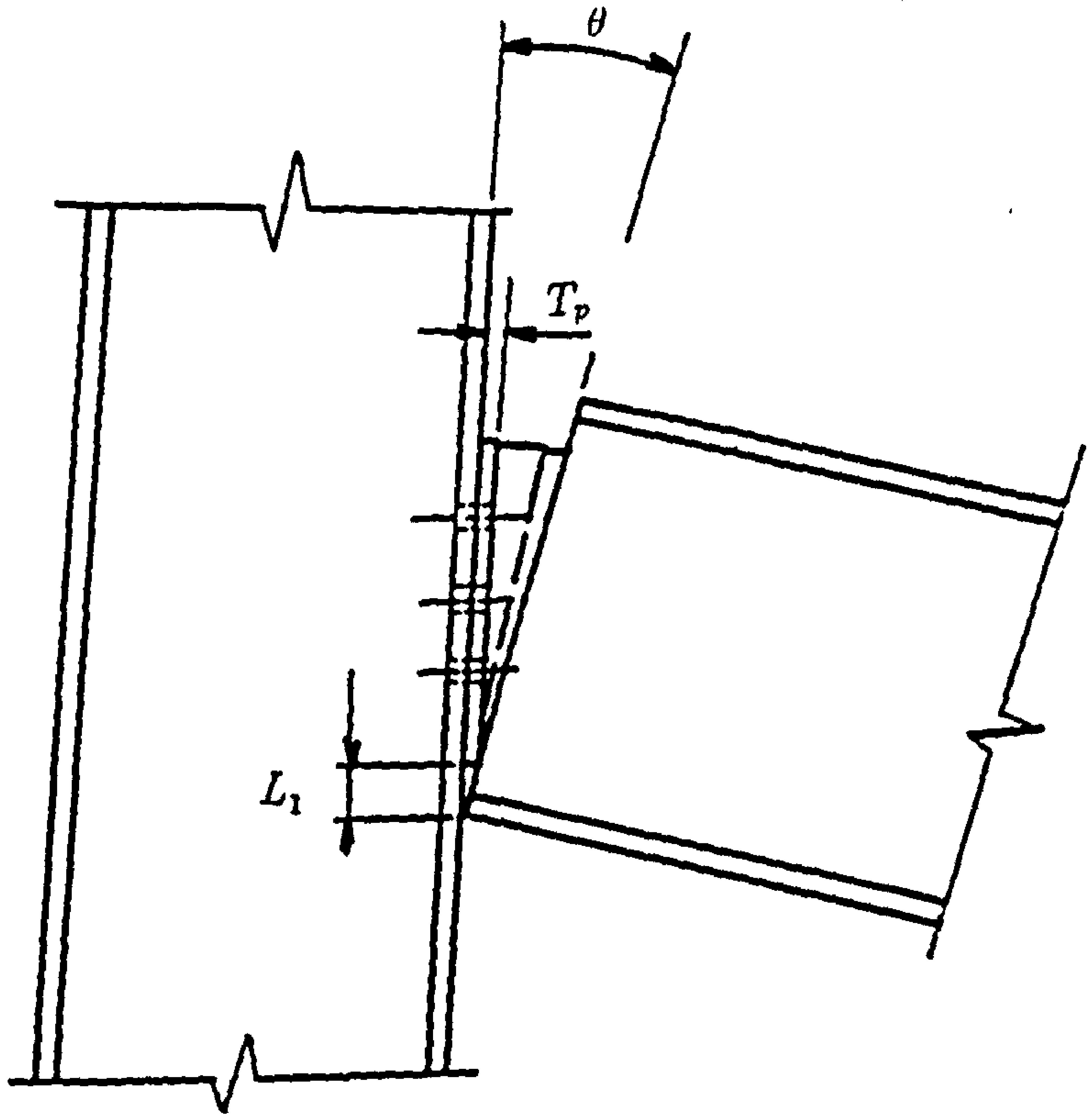


Figure 2.3 Beam flange bearing against column.

FIG.2-4 Moment-rotation curves for header plate connections of reference [2.12].

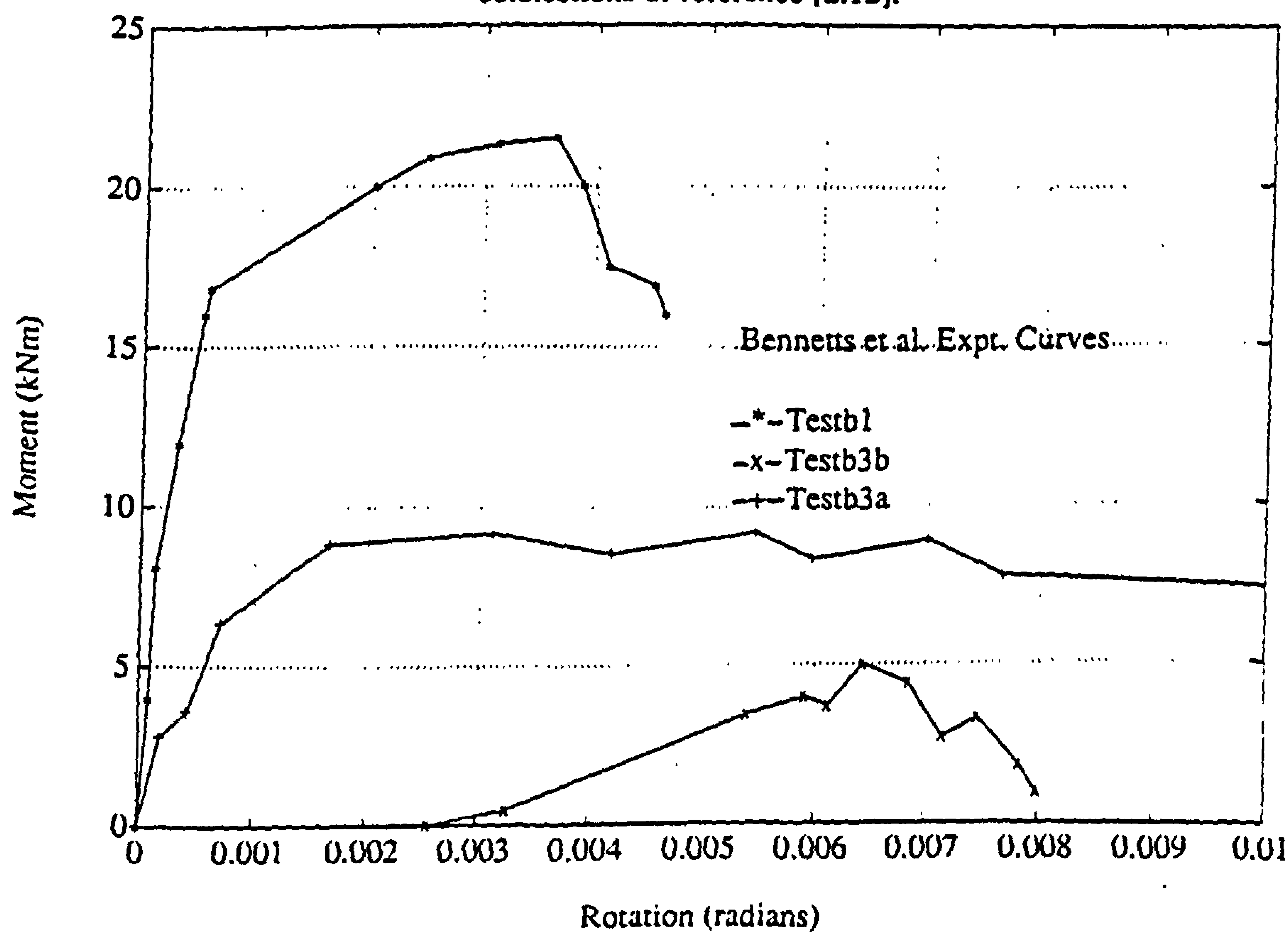


FIG.2-5 Effect of end-plate thickness on moment-rotation characteristics (a)

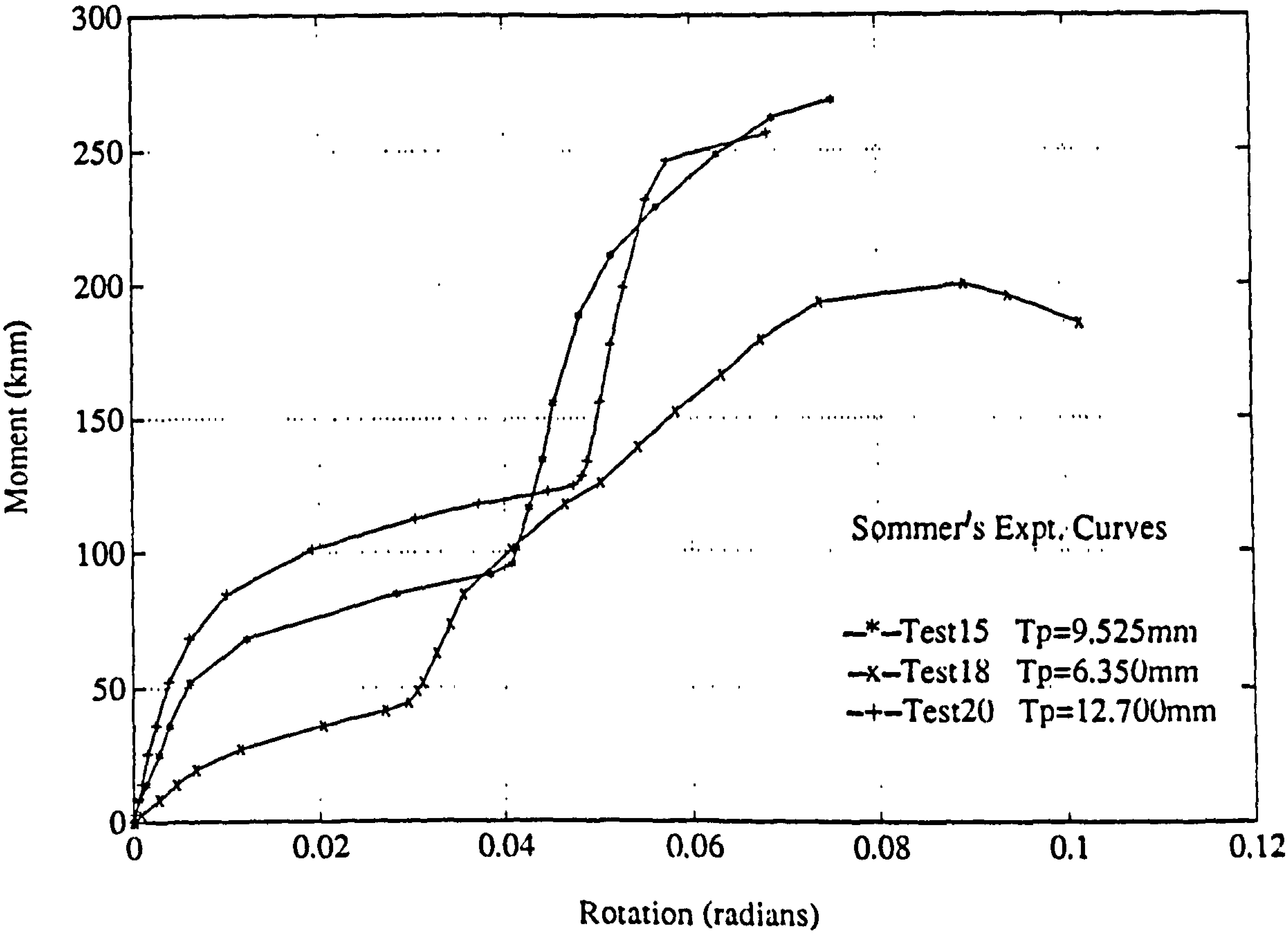


FIG.2-5 Effect of end-plate thickness on moment-rotation characteristics (b)

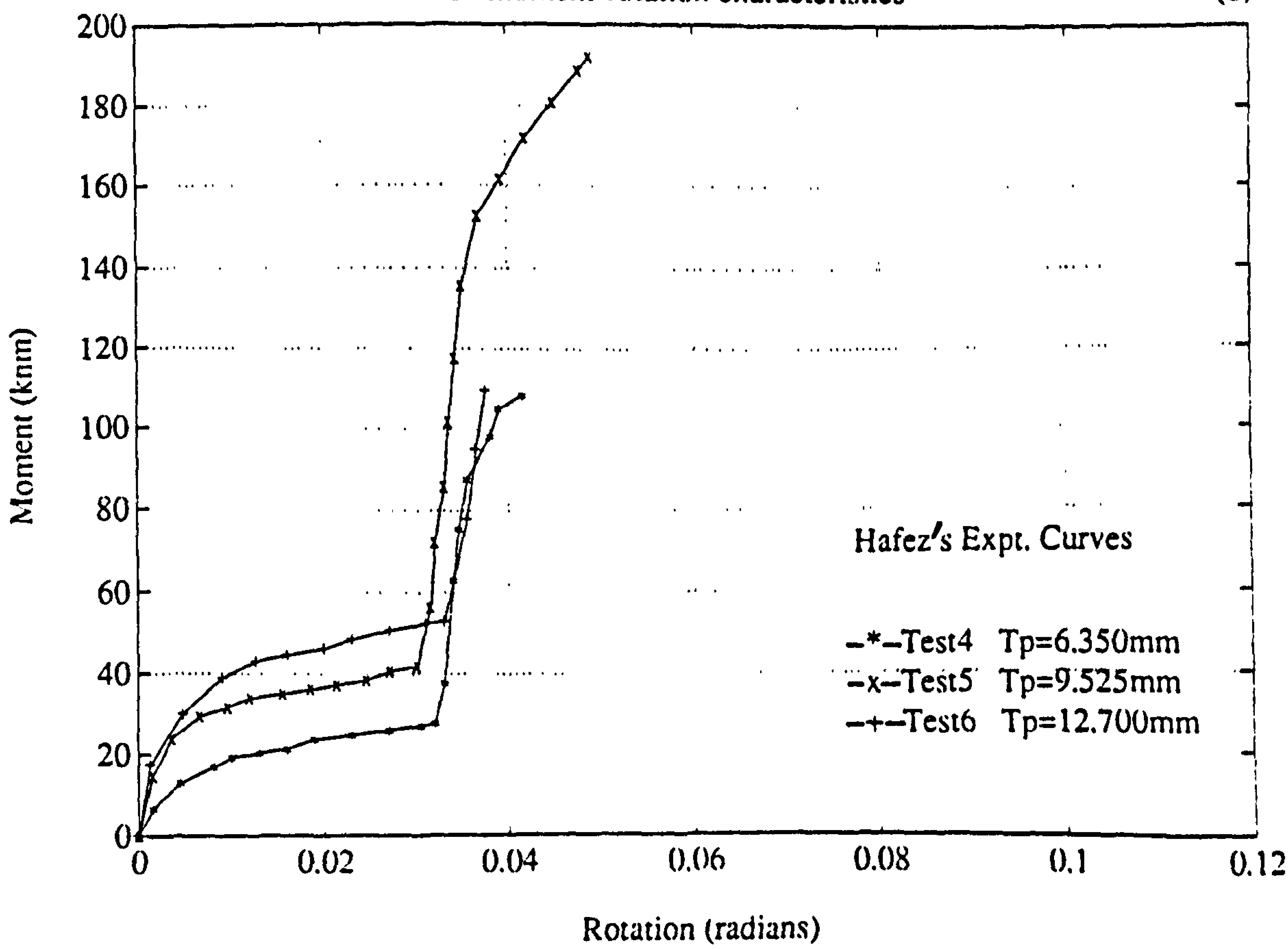


FIG.2-5 Effect of end-plate thickness
on moment-rotation characteristics (c)

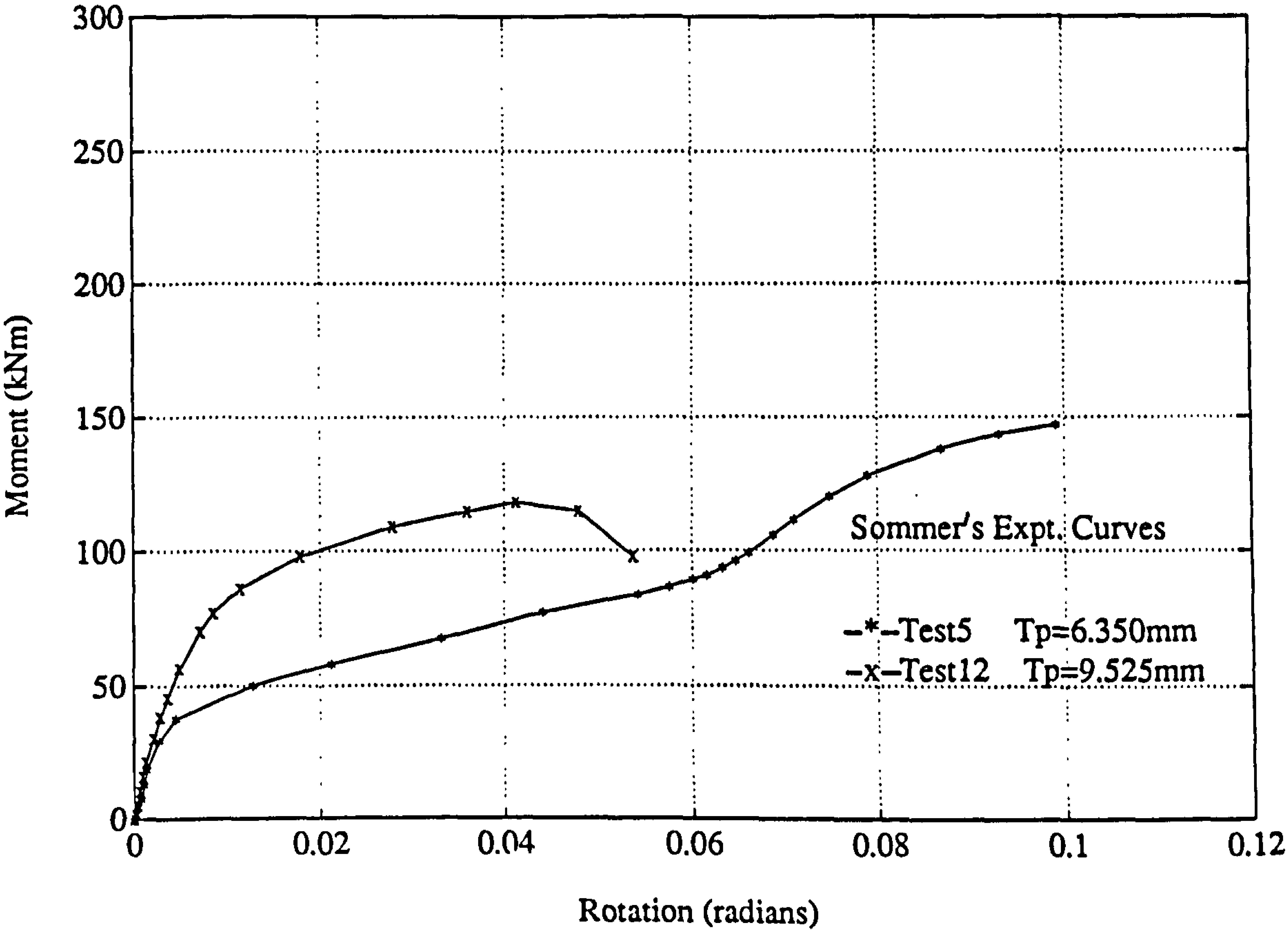


FIG.2-5 Effect of end-plate thickness
on moment-rotation characteristics (d)

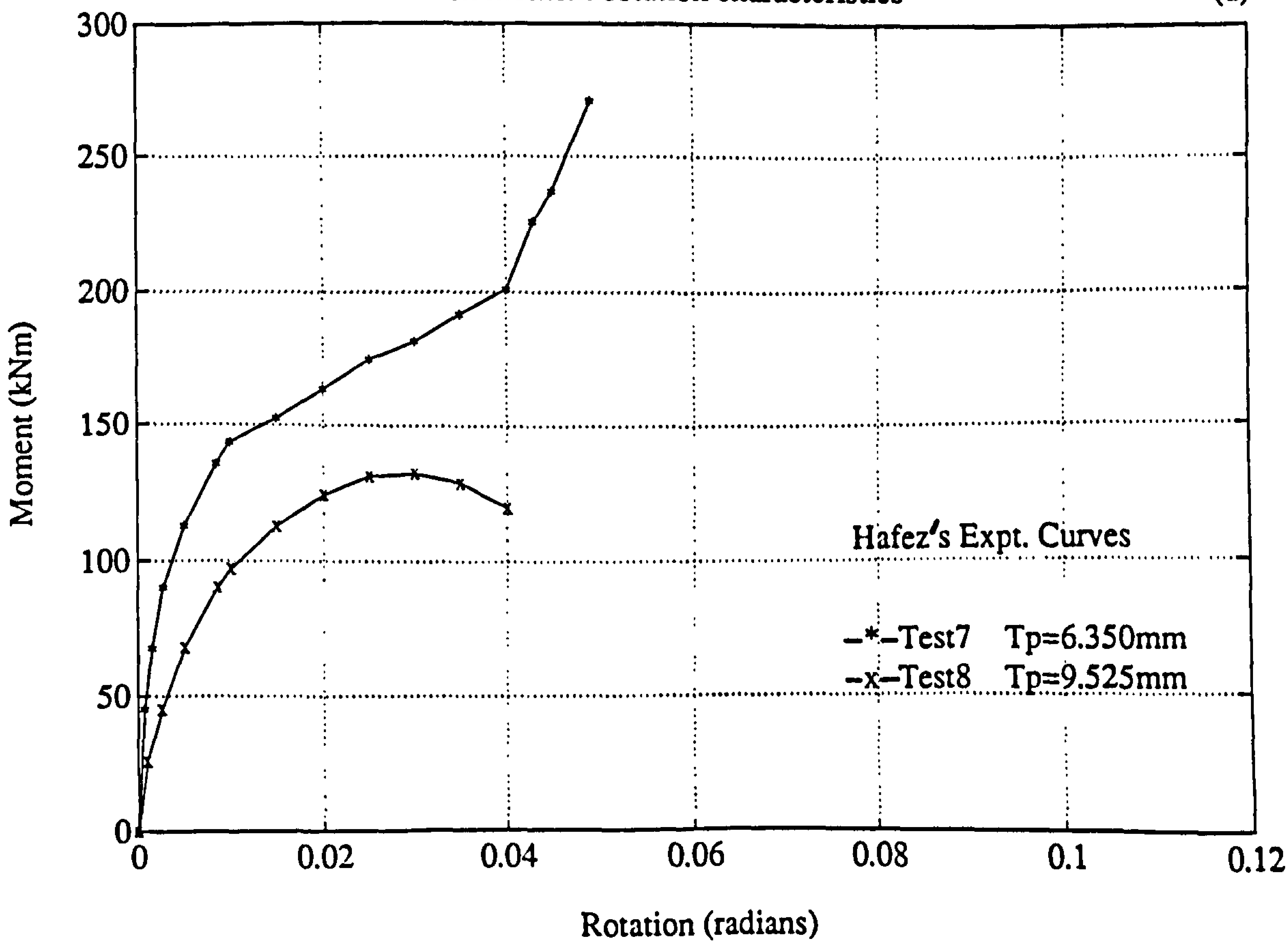


FIG.2-6 Effect of connection depth on moment-rotation characteristics (a)

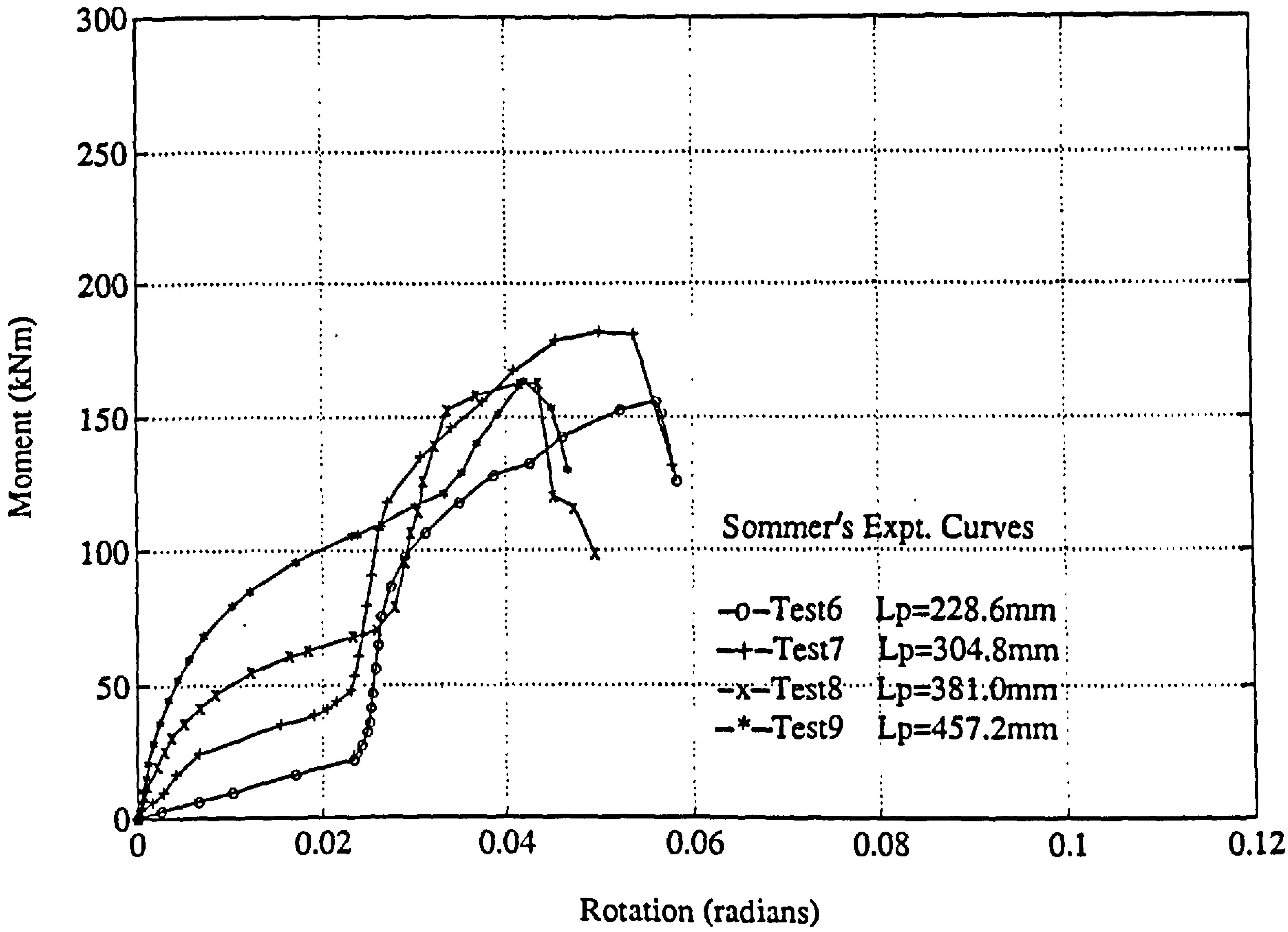


FIG.2-6 Effect of connection depth on moment-rotation characteristics (b)

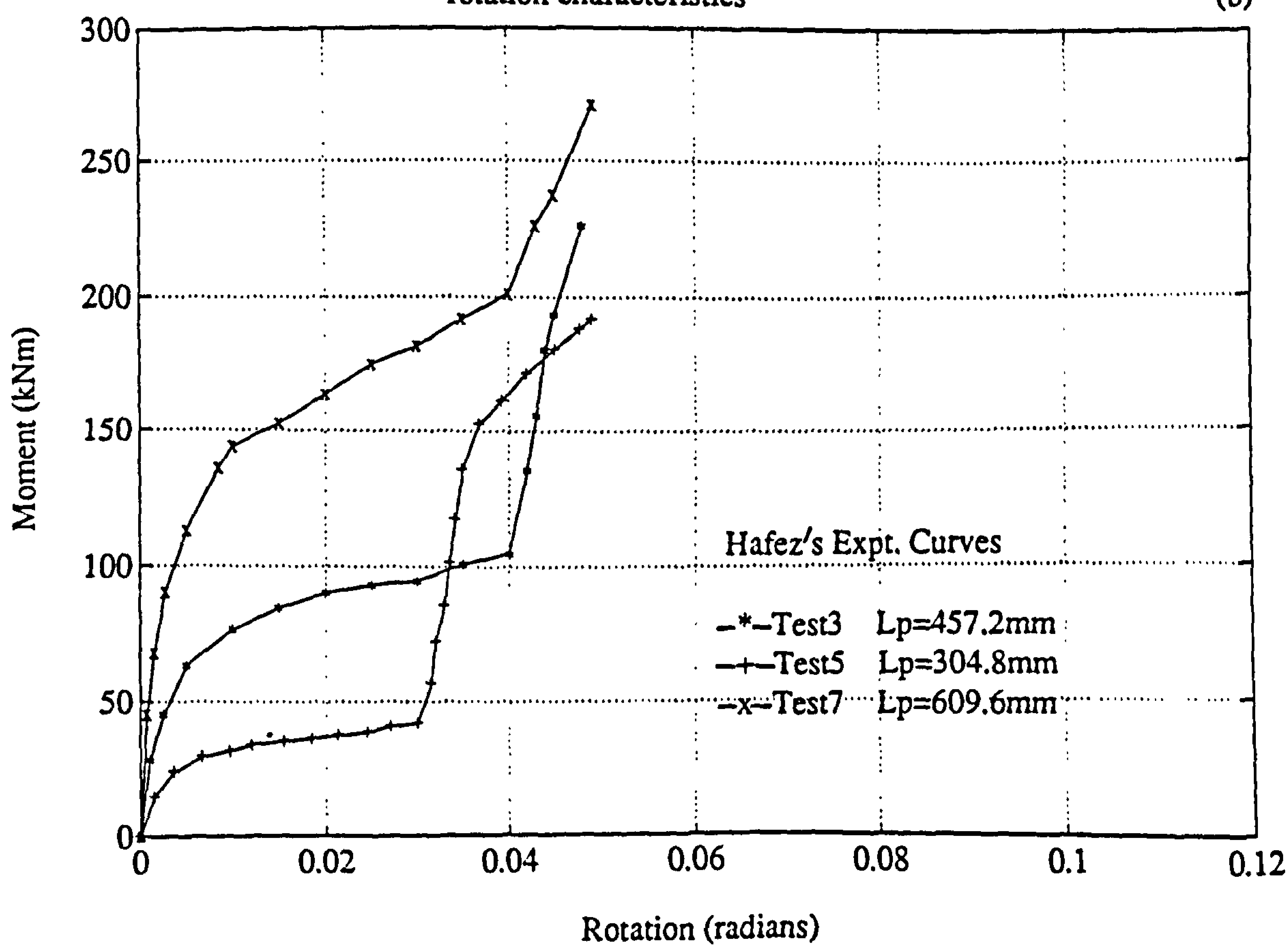


FIG.2-7 Effect of beam depth on moment-rotation characteristics

(a)

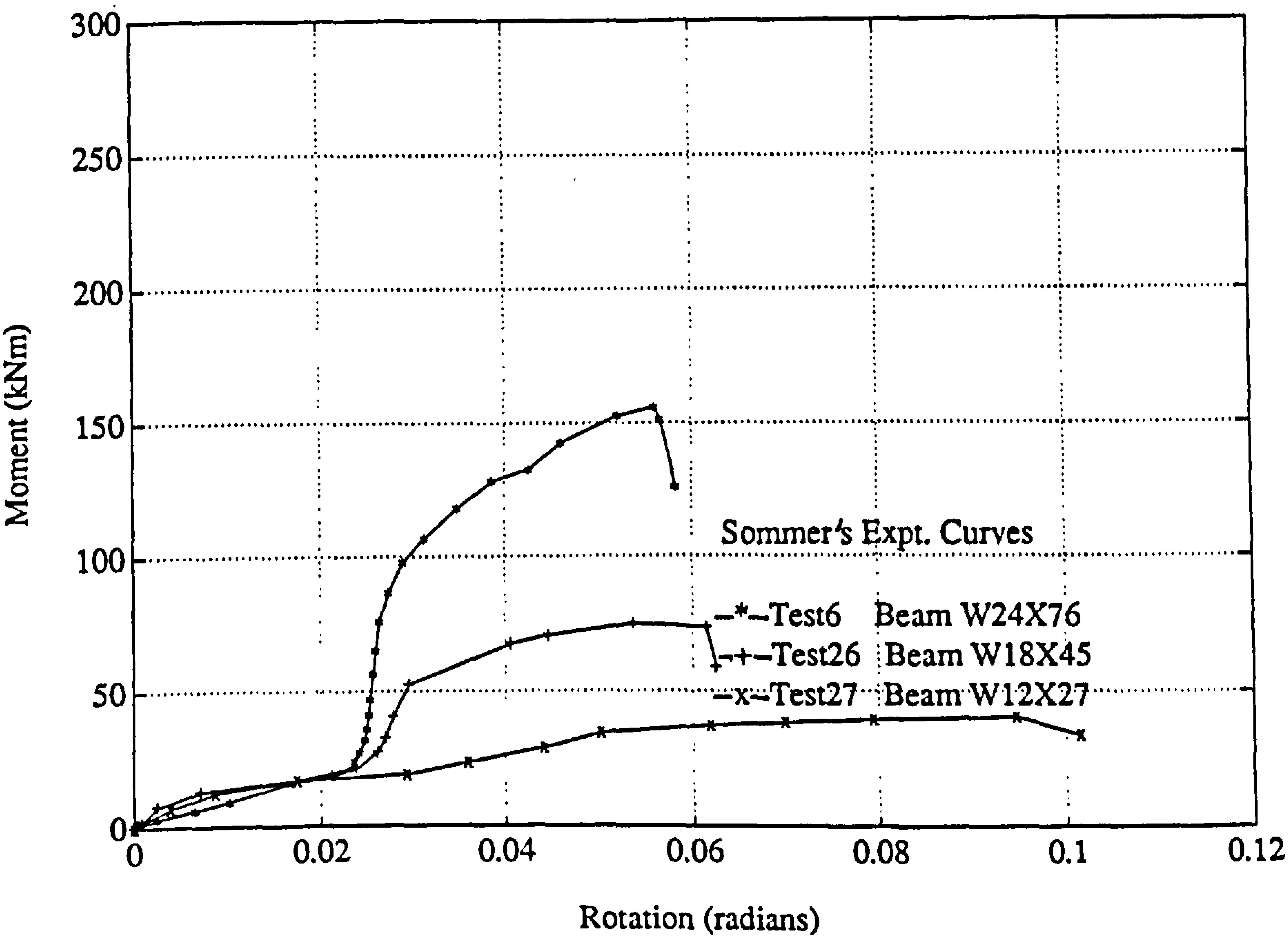


FIG.2-7 Effect of beam depth on moment-rotation characteristics

(b)

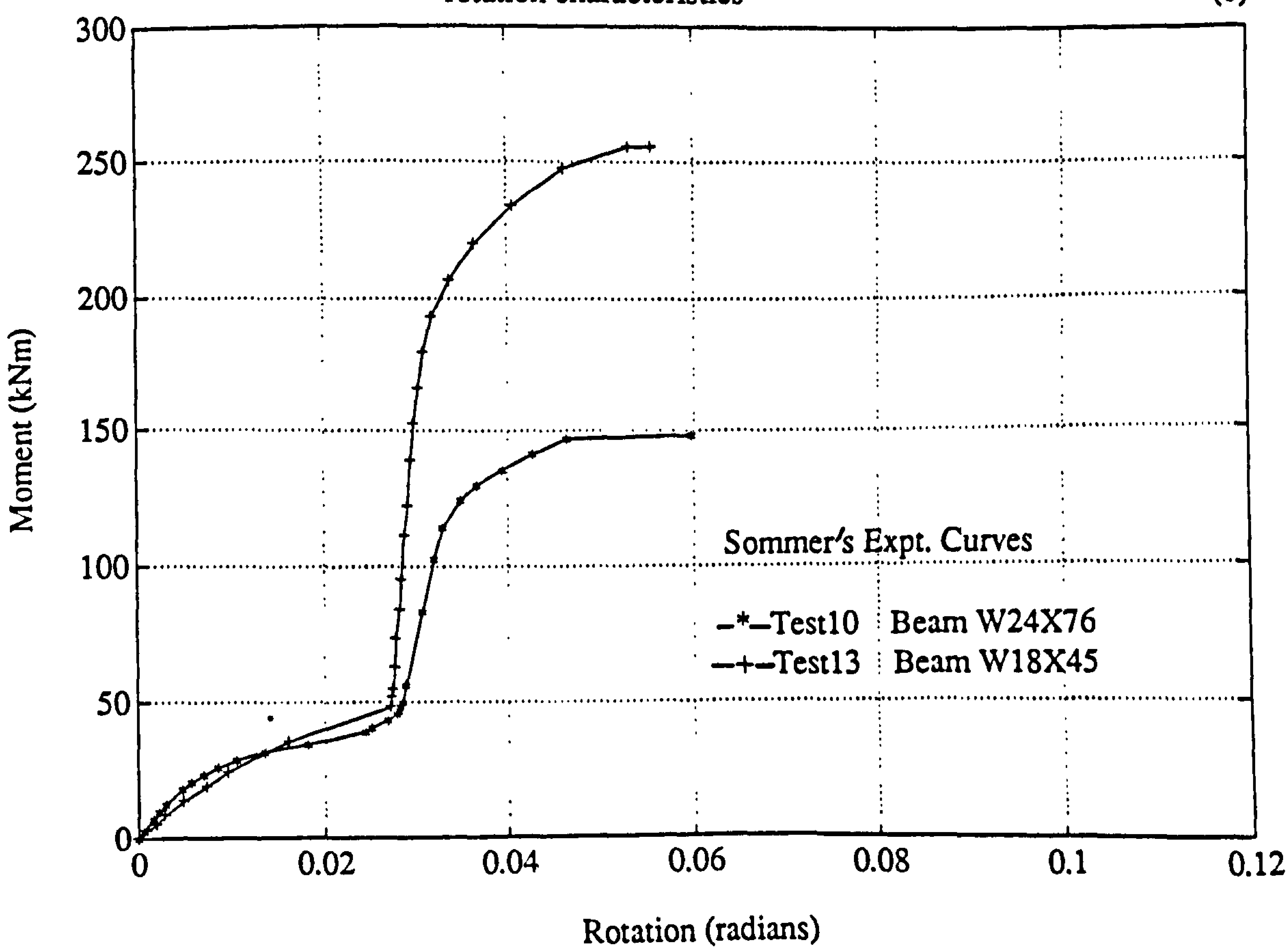


FIG.2-8 Effect of gauge distance on moment-rotation characteristics (a)

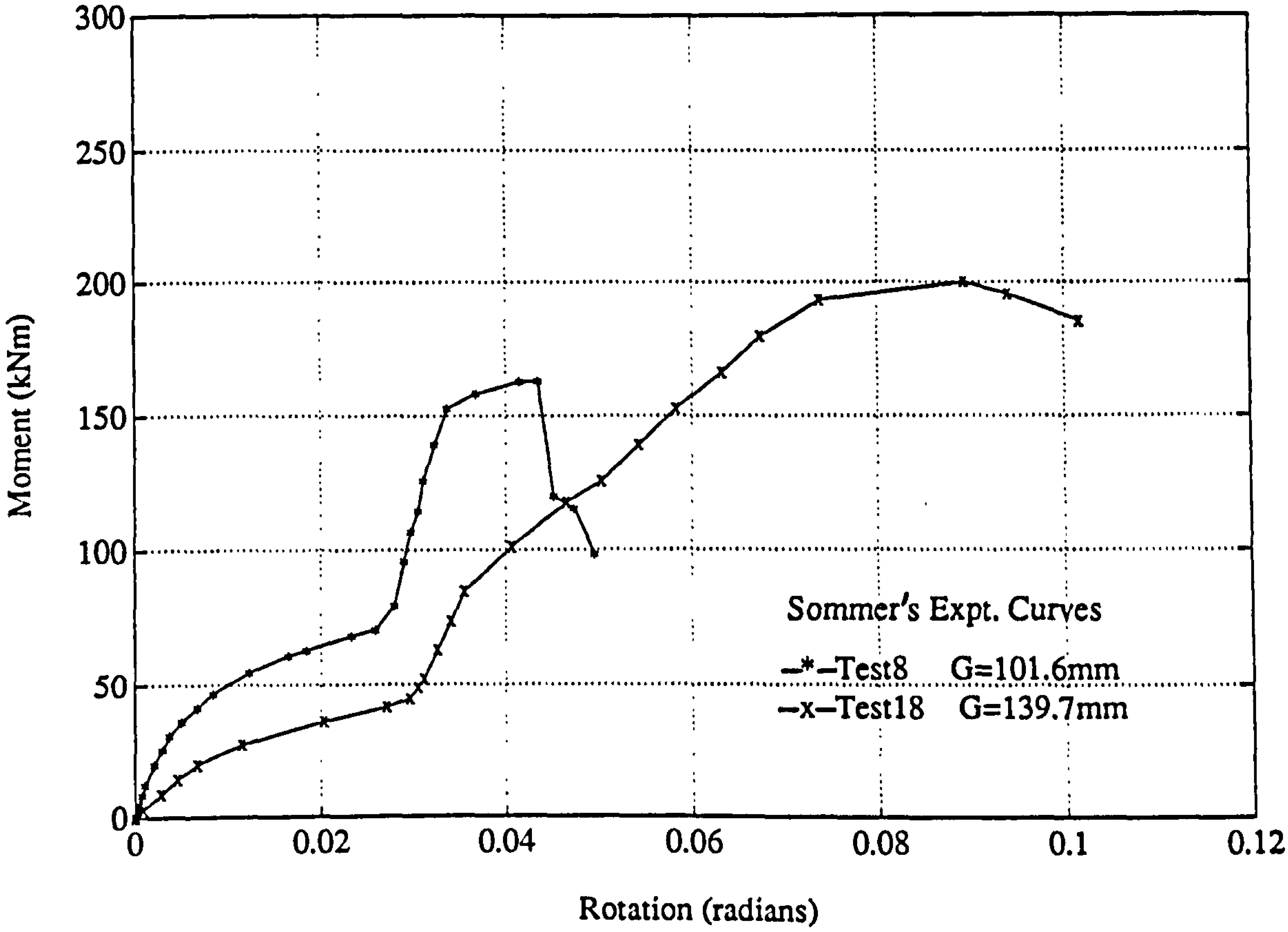


FIG.2-8 Effect of gauge distance on moment-rotation characteristics (b)

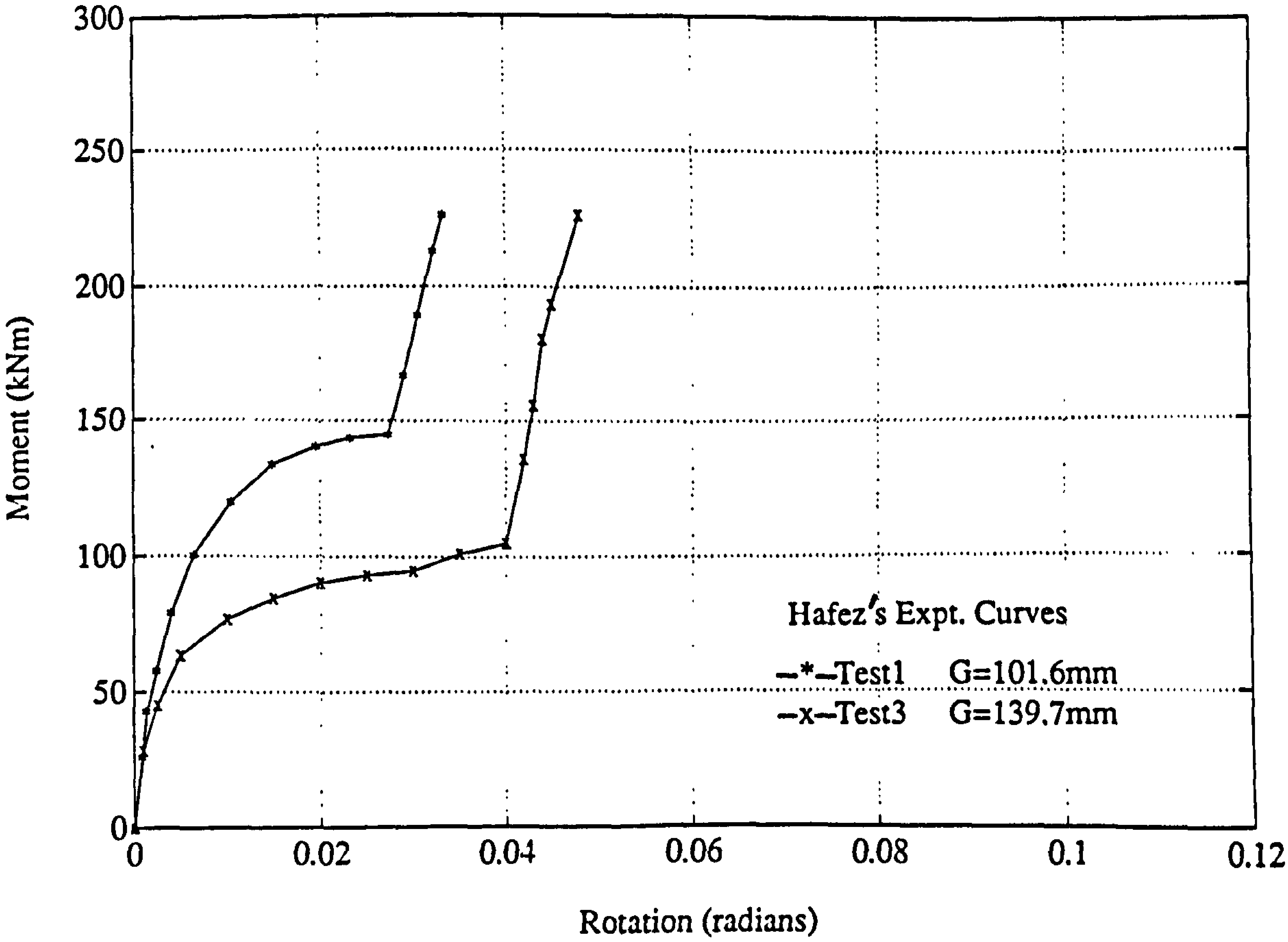
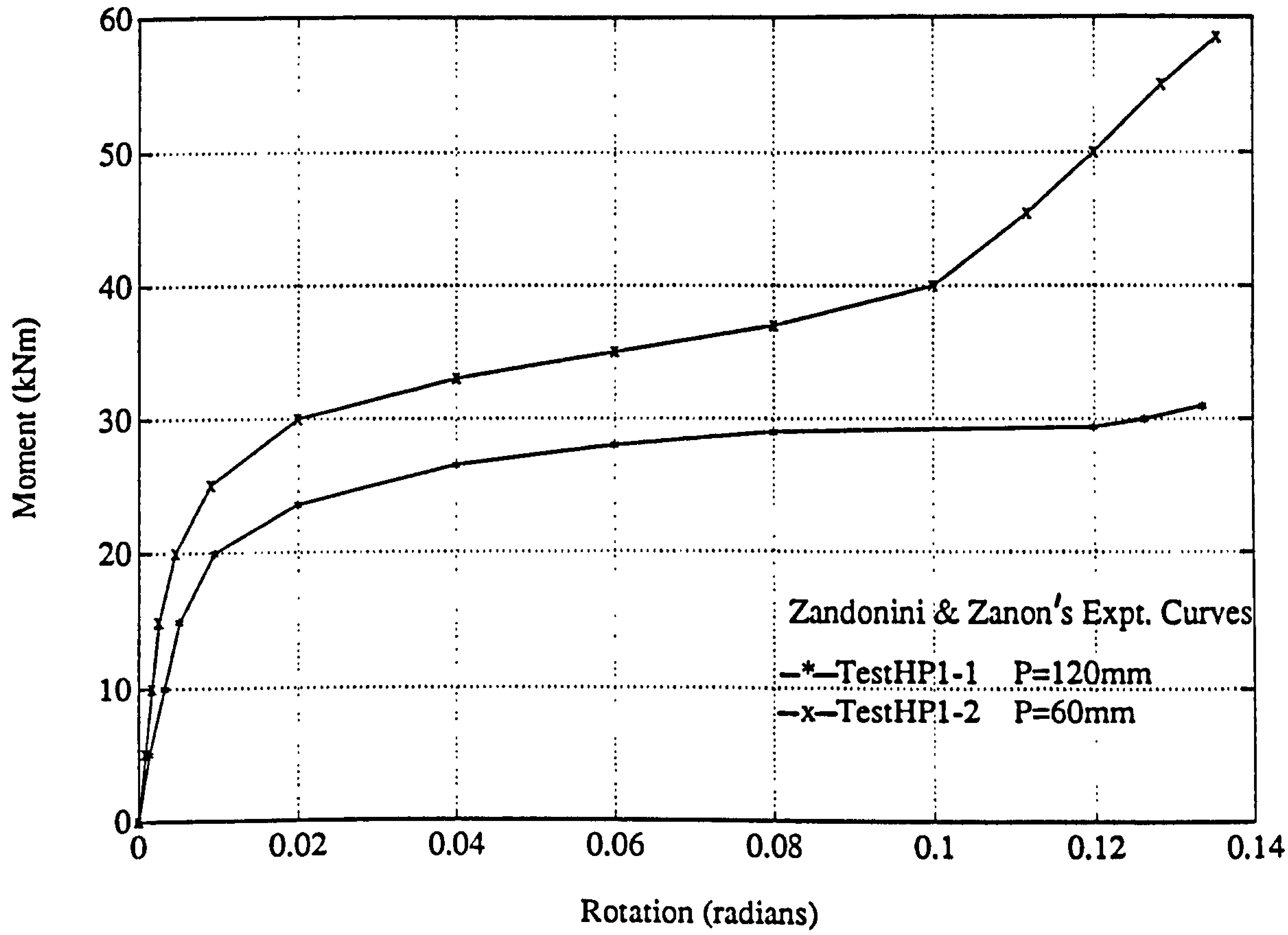


FIG.2-9 Effect of pitch distance on moment-rotation characteristics



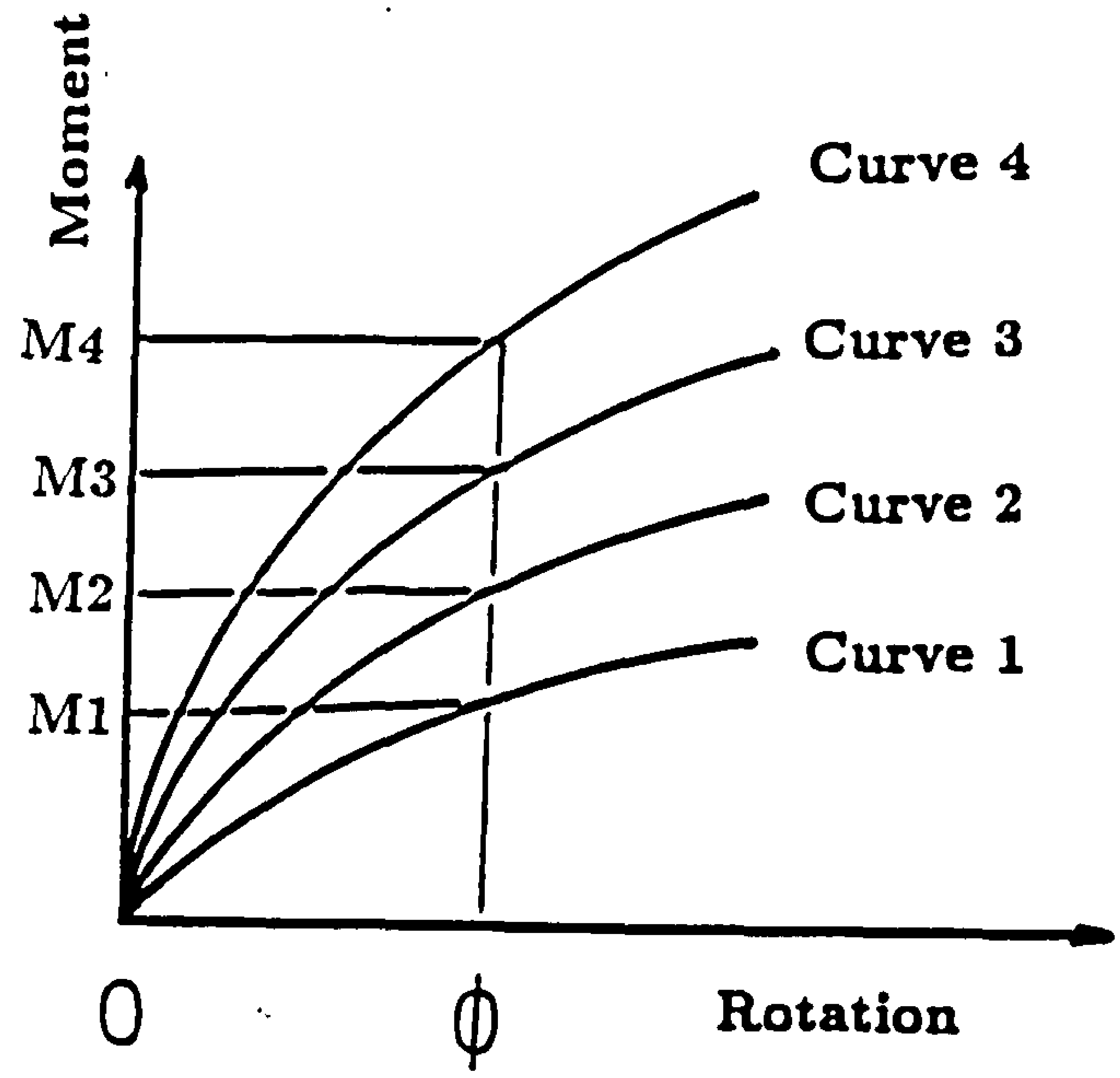


Figure 2.10 Family of moment-rotation curves varying in parameter q_j.

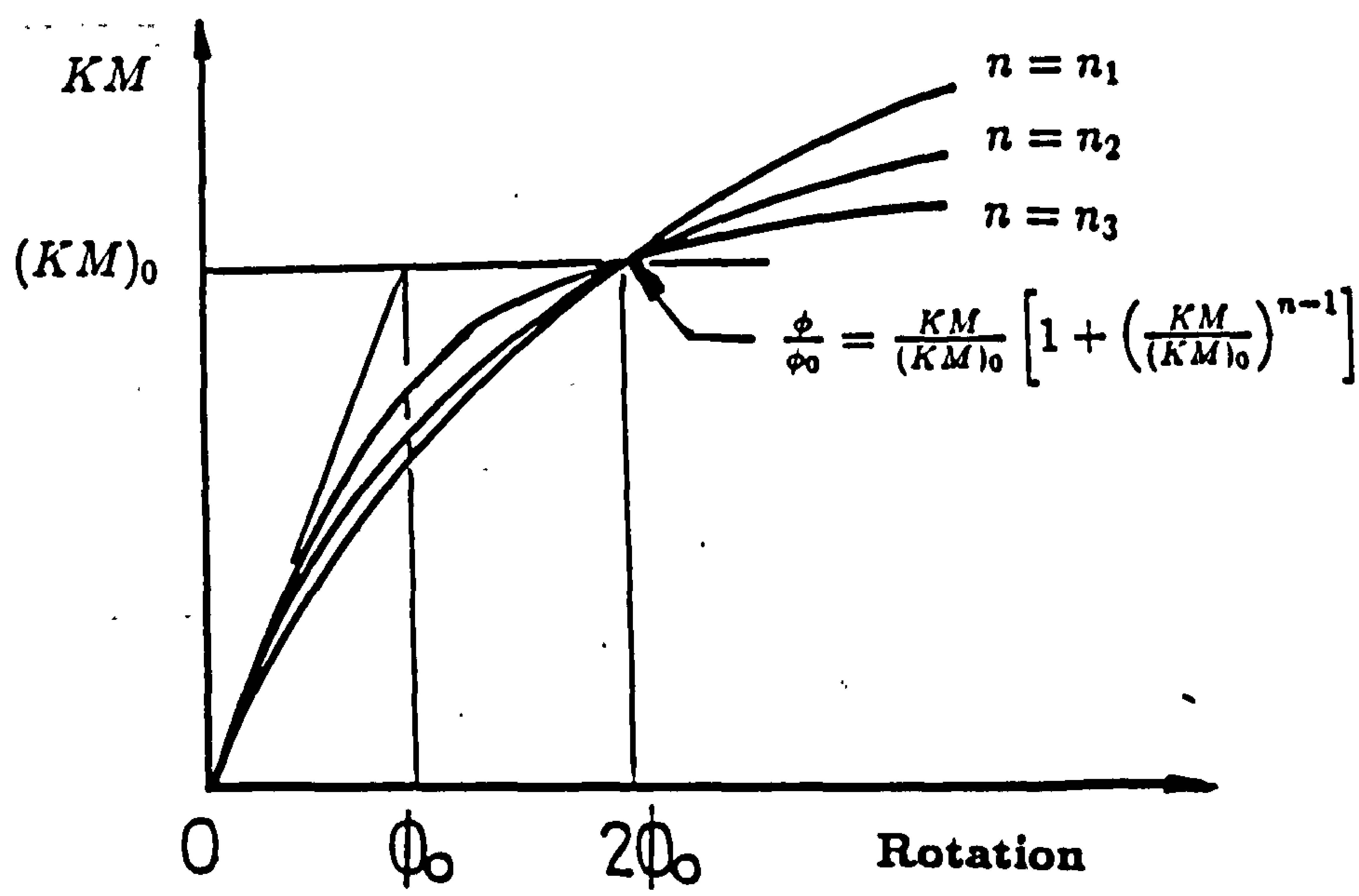


Figure 2.11 Ramberg-Osgood function for standardized moment-rotation curves

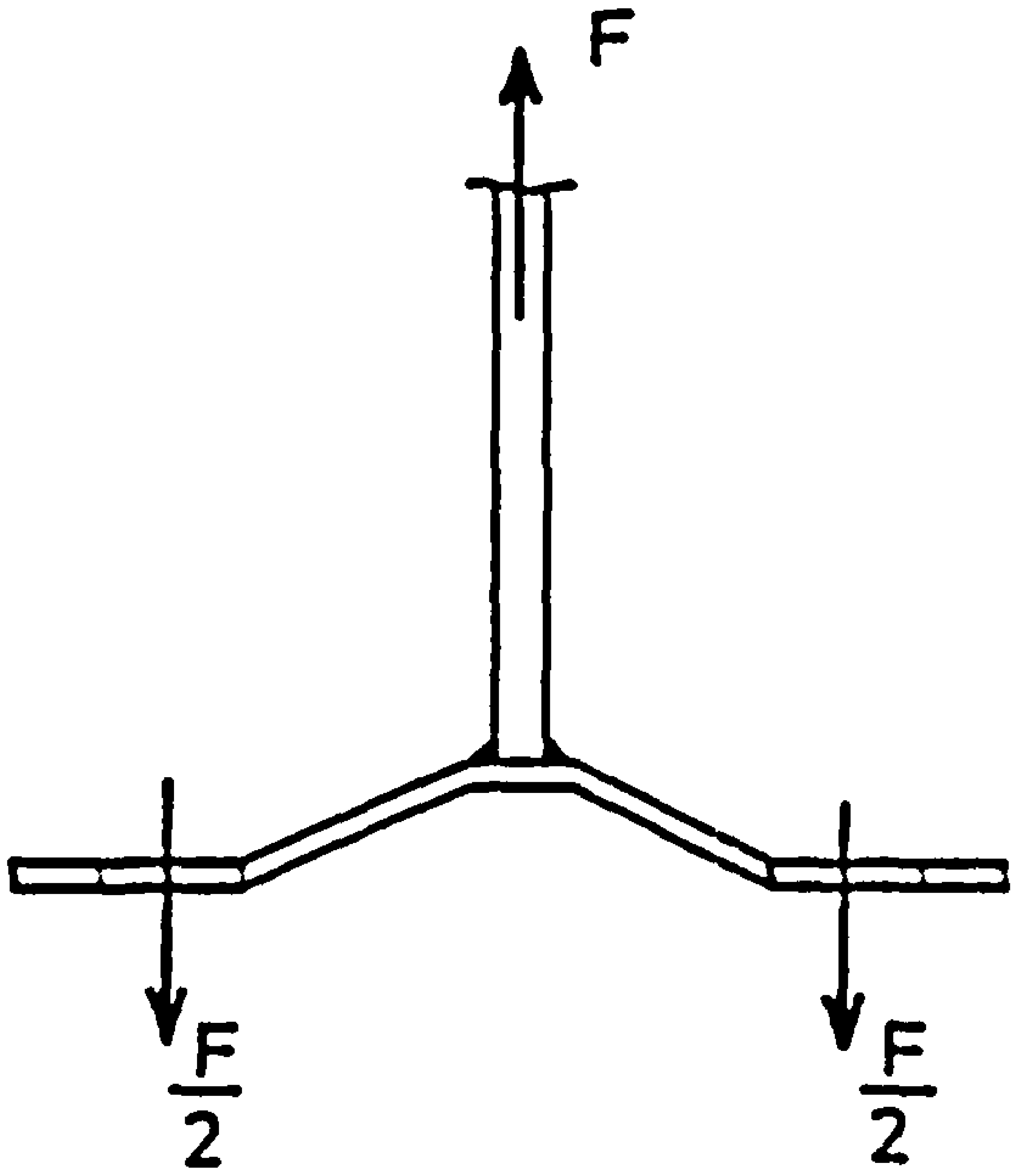


Figure 2.12 Tee-stub model used in the analysis of flexible end-plate [2.23].

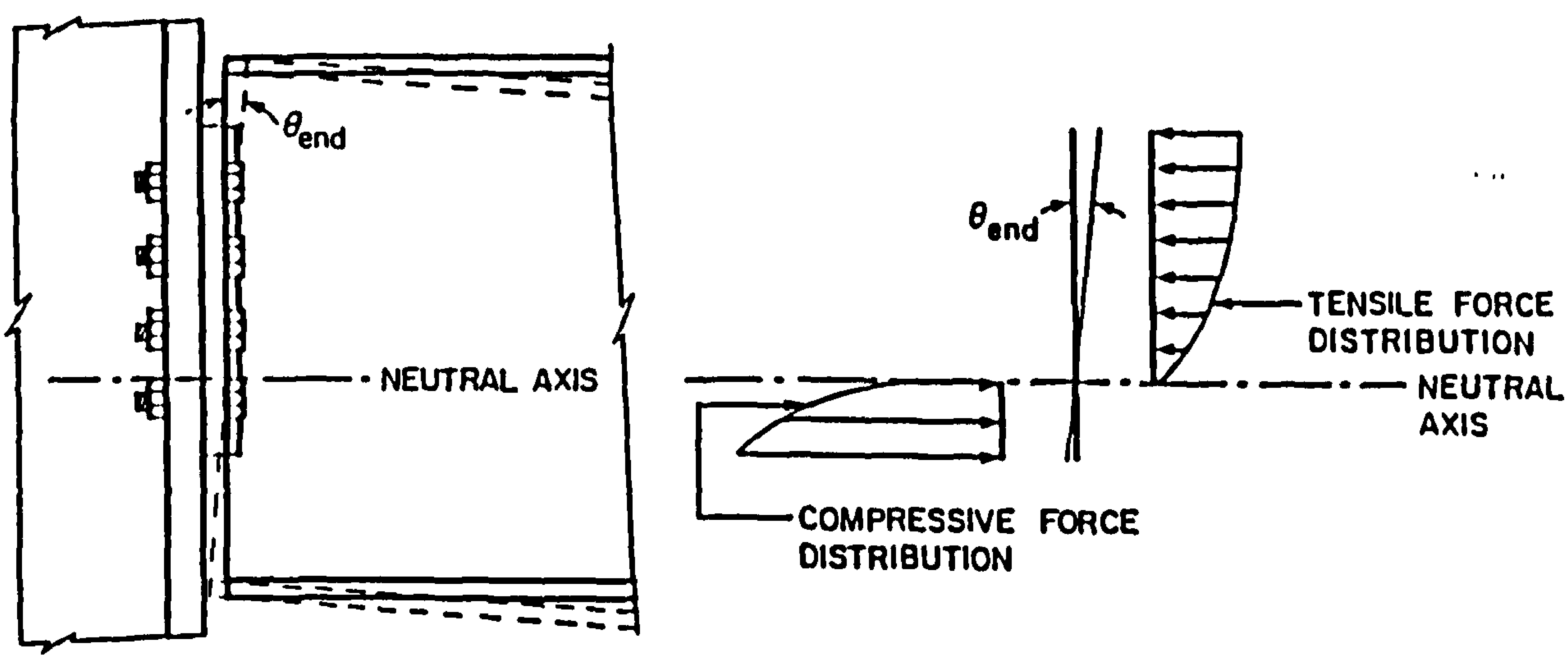


Figure 2.13 Flexural force distribution at flexible end-plate connection [2.23].

FIG.2-14 Comparison between analytical moment-rotation relationships and Sommer's test 5

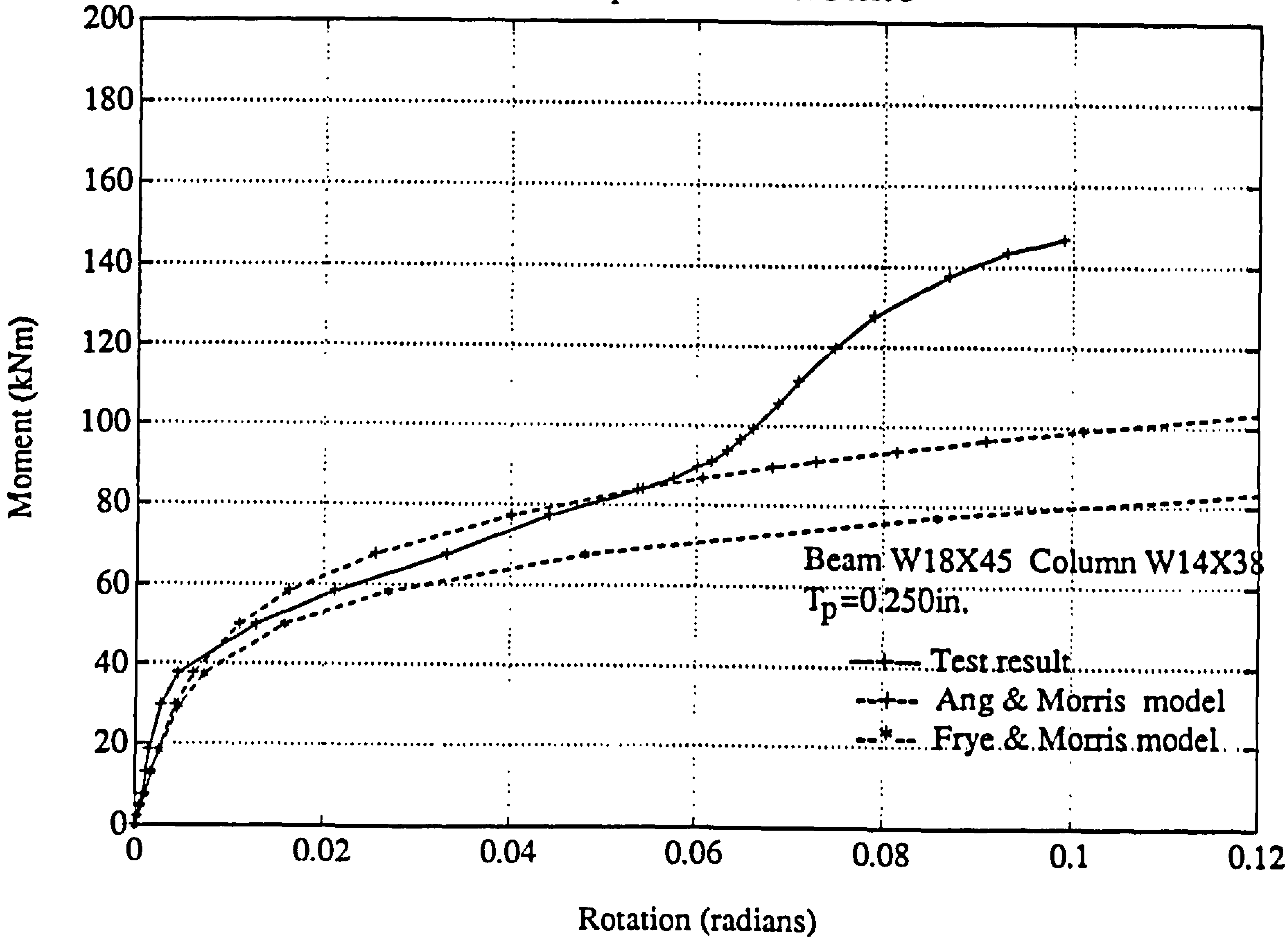


FIG.2-15 Comparison between analytical moment-rotation relationships and Sommer's test 6

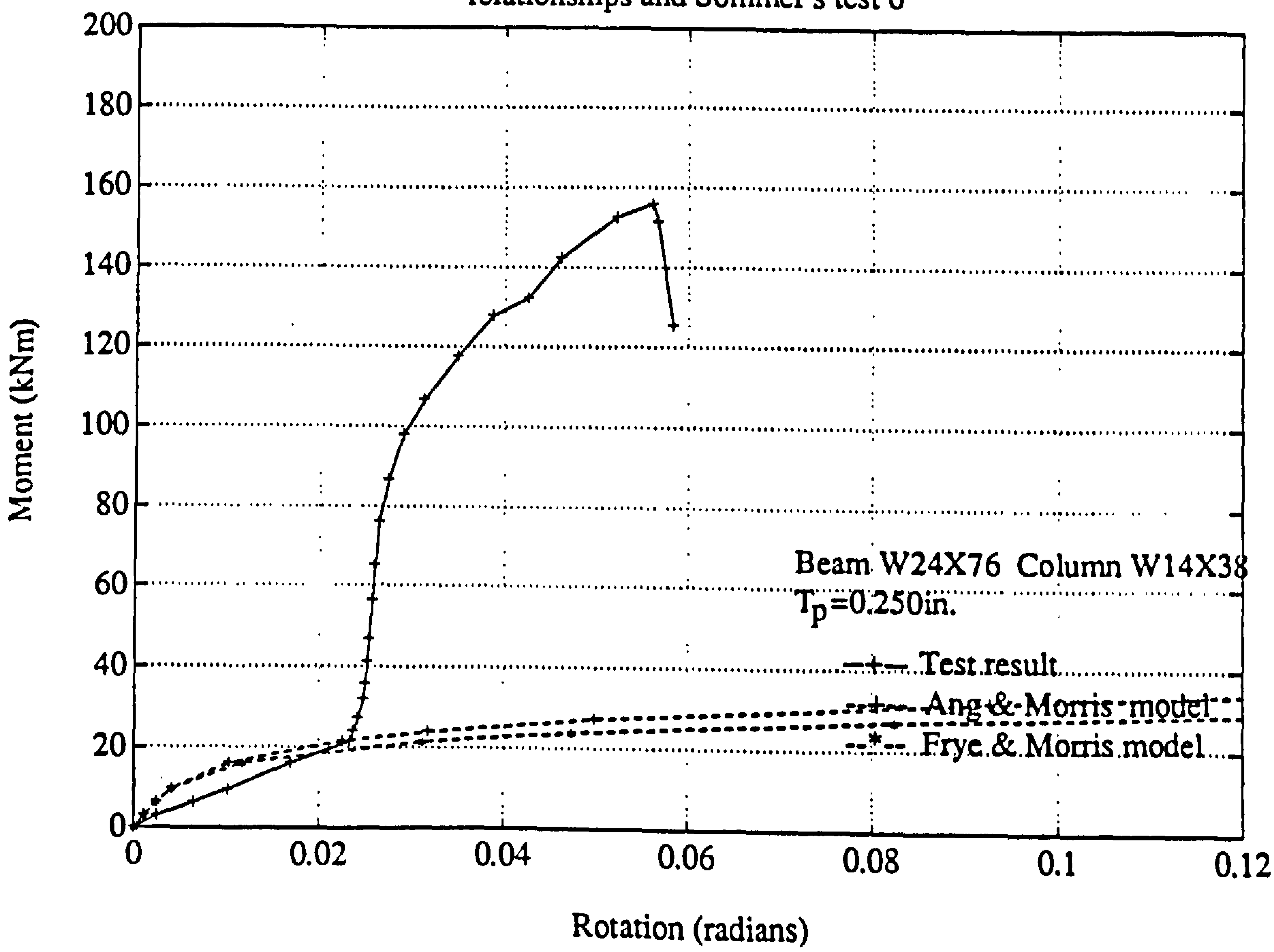


FIG.2-16 Comparison between analytical moment-rotation relationships and Sommer's test 7

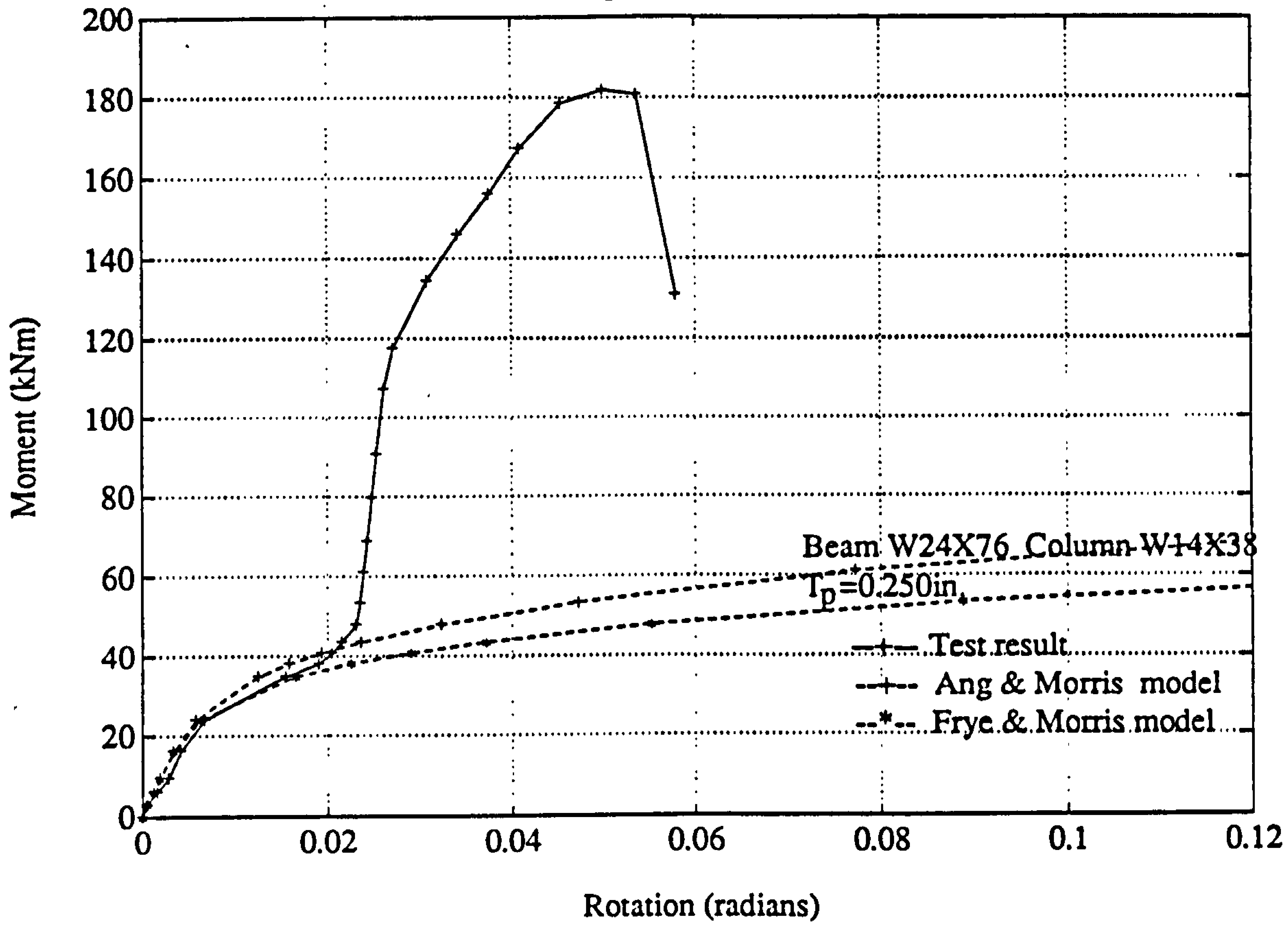


FIG.2-17 Comparison between analytical moment-rotation relationships and Sommer's test 8

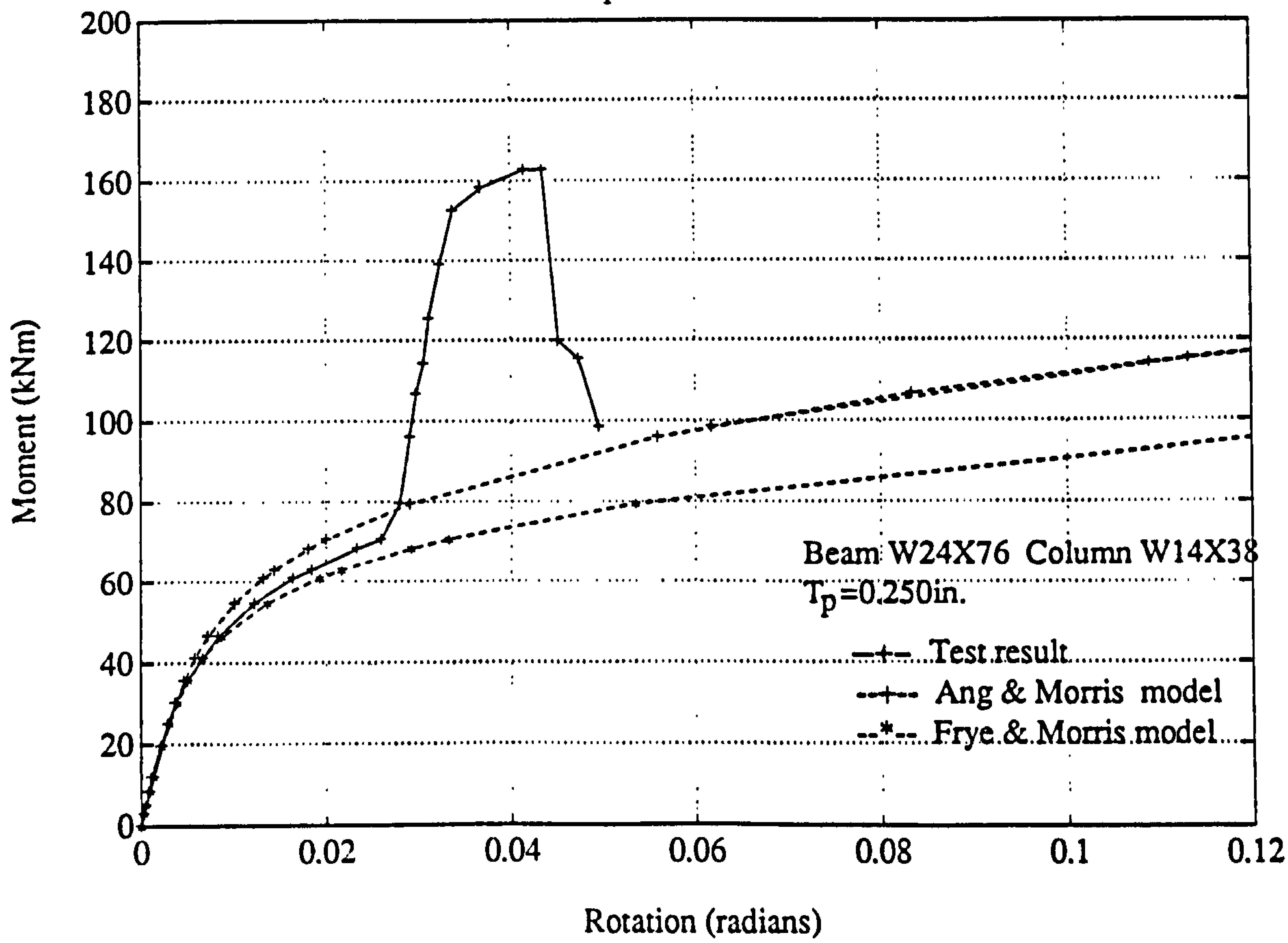


FIG.2-18 Comparison between analytical moment-rotation relationships and Sommer's test 9

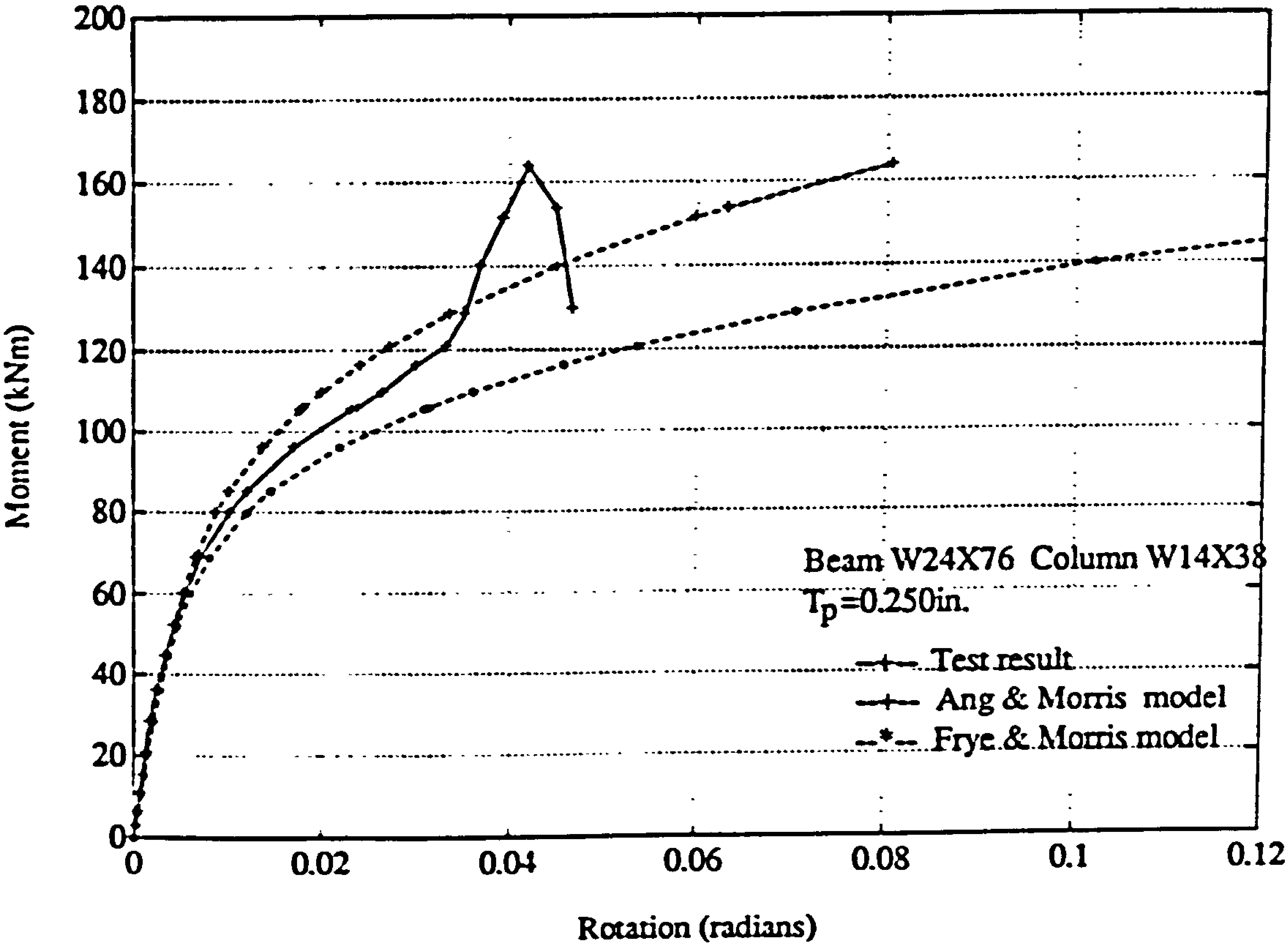


FIG.2-19 Comparison between analytical moment-rotation relationships and Sommer's test 10

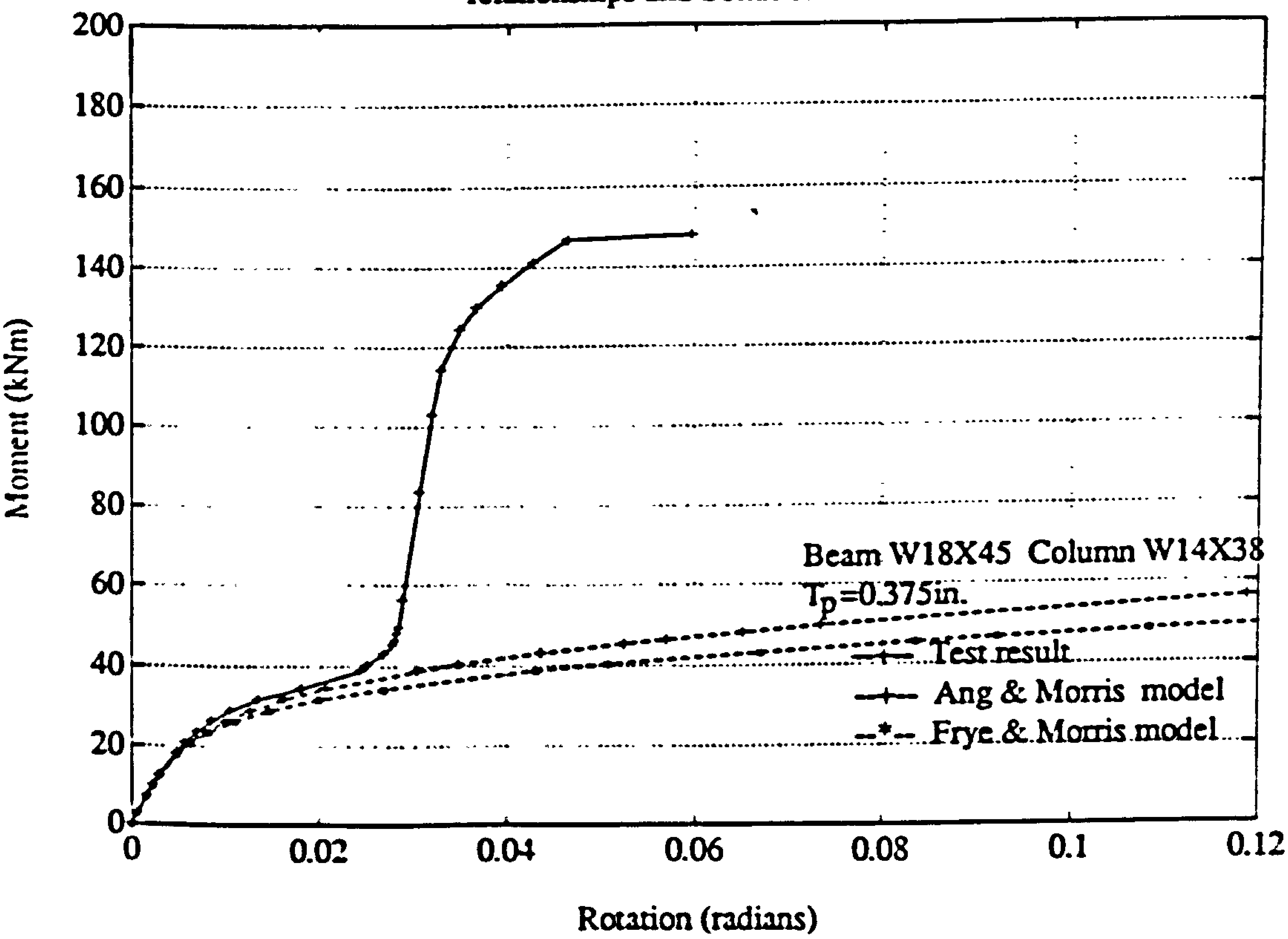


FIG.2-20 Comparison between analytical moment-rotation relationships and Sommer's test 11

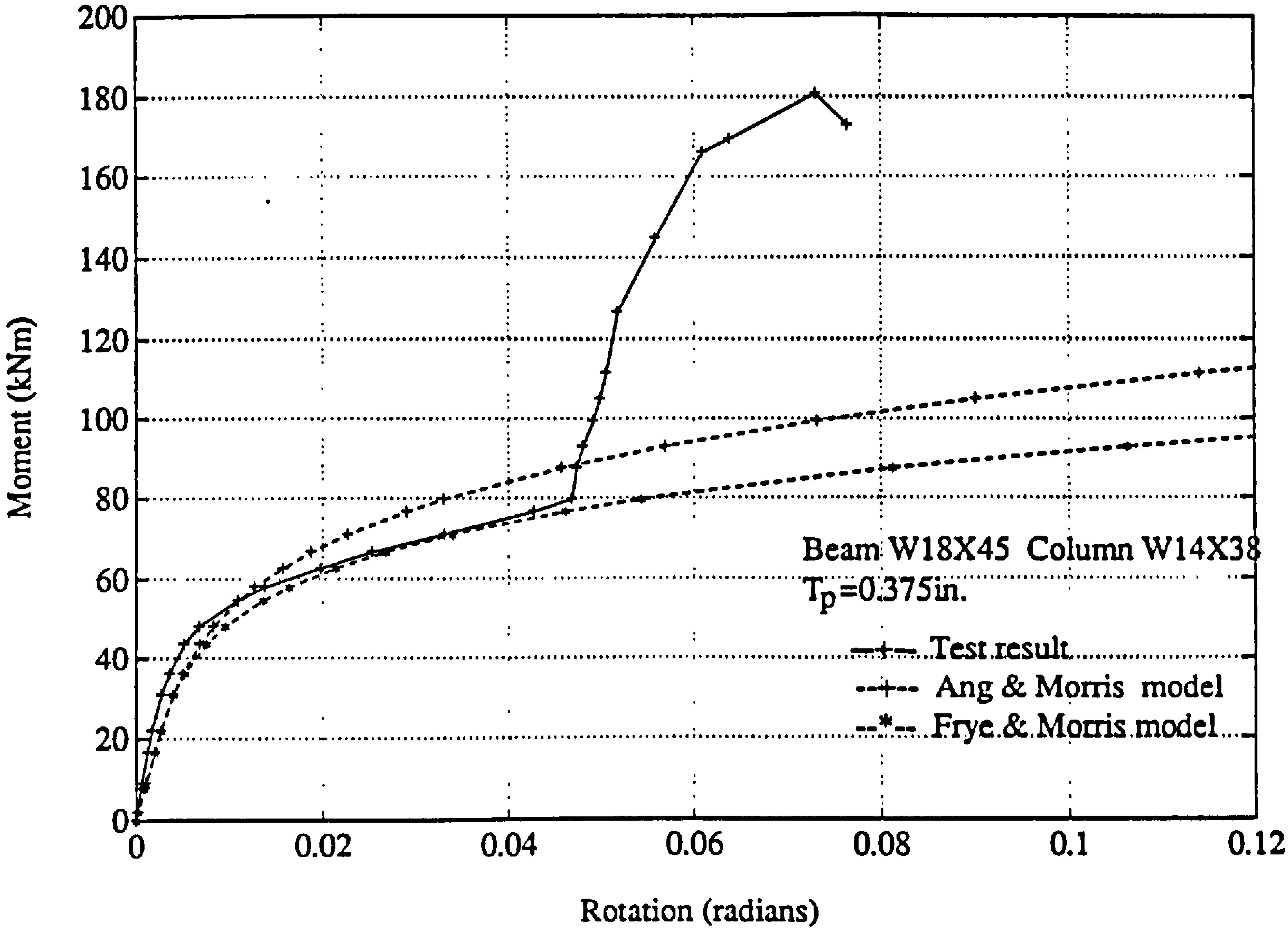


FIG.2-21 Comparison between analytical moment-rotation relationships and Sommer's test 12

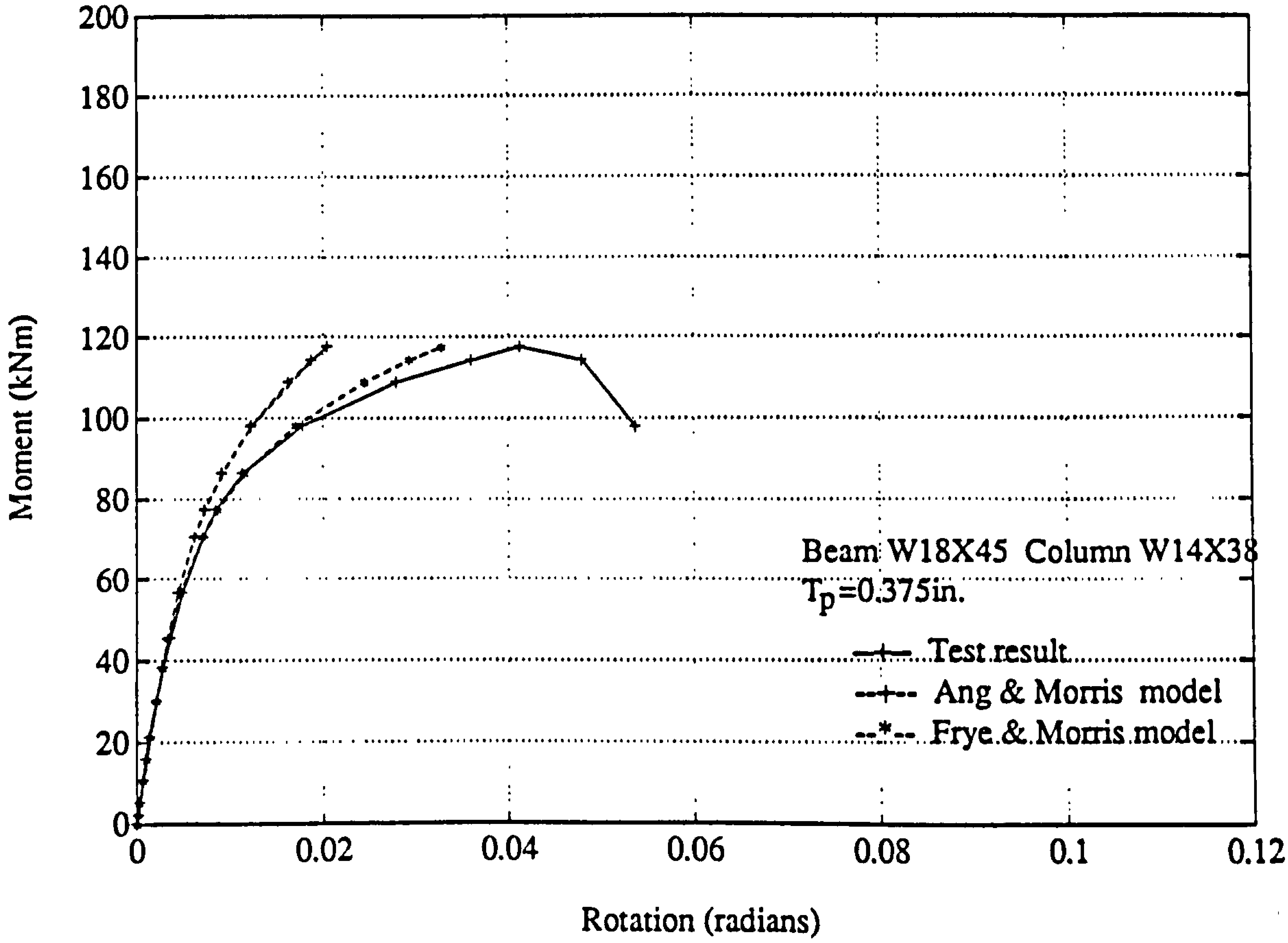


FIG.2-22 Comparison between analytical moment-rotation relationships and Sommer's test 13

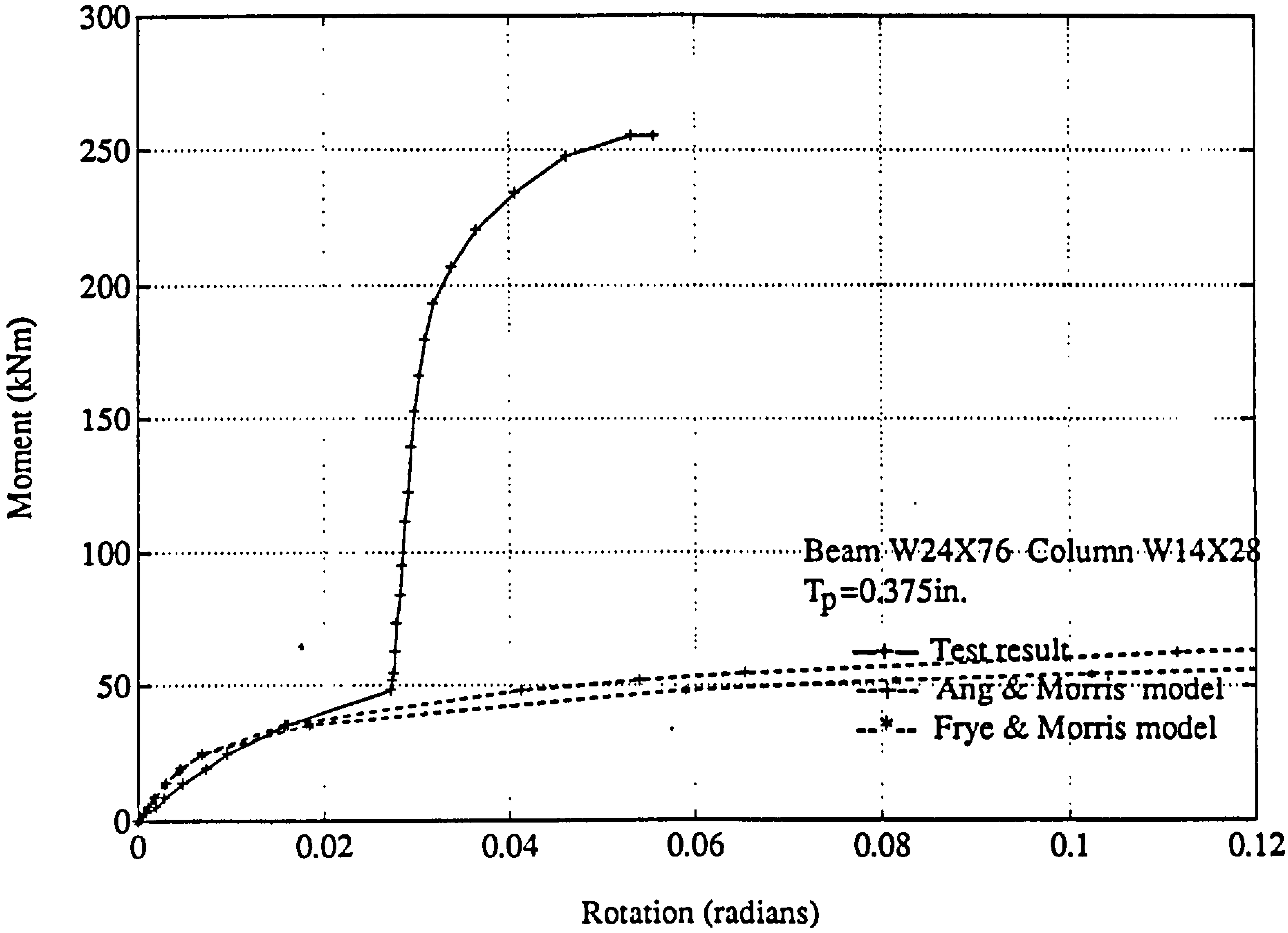


FIG.2-23 Comparison between analytical moment-rotation relationships and Sommer's test 14

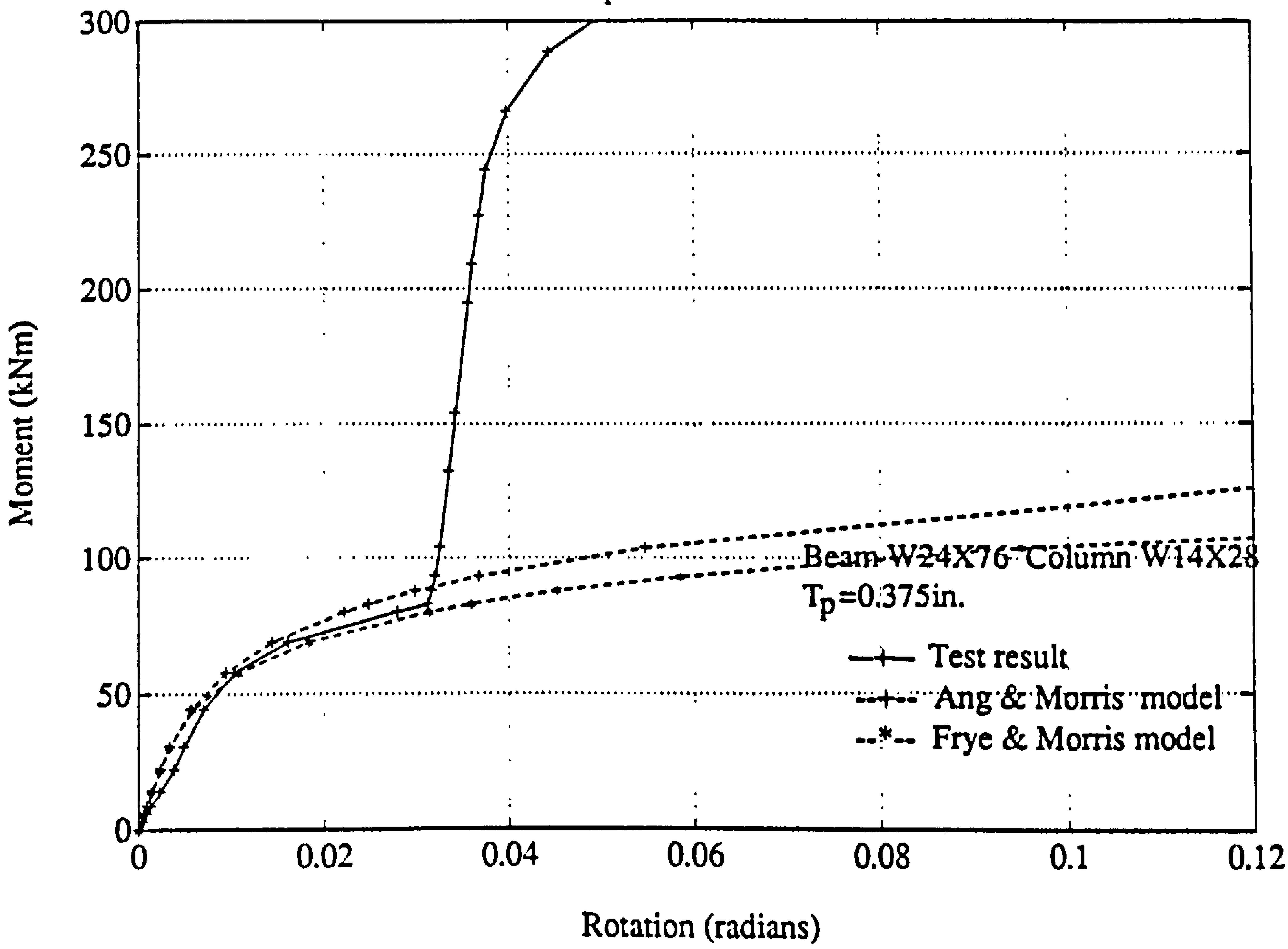


FIG.2-24 Comparison between analytical moment-rotation relationships and Sommer's test 15

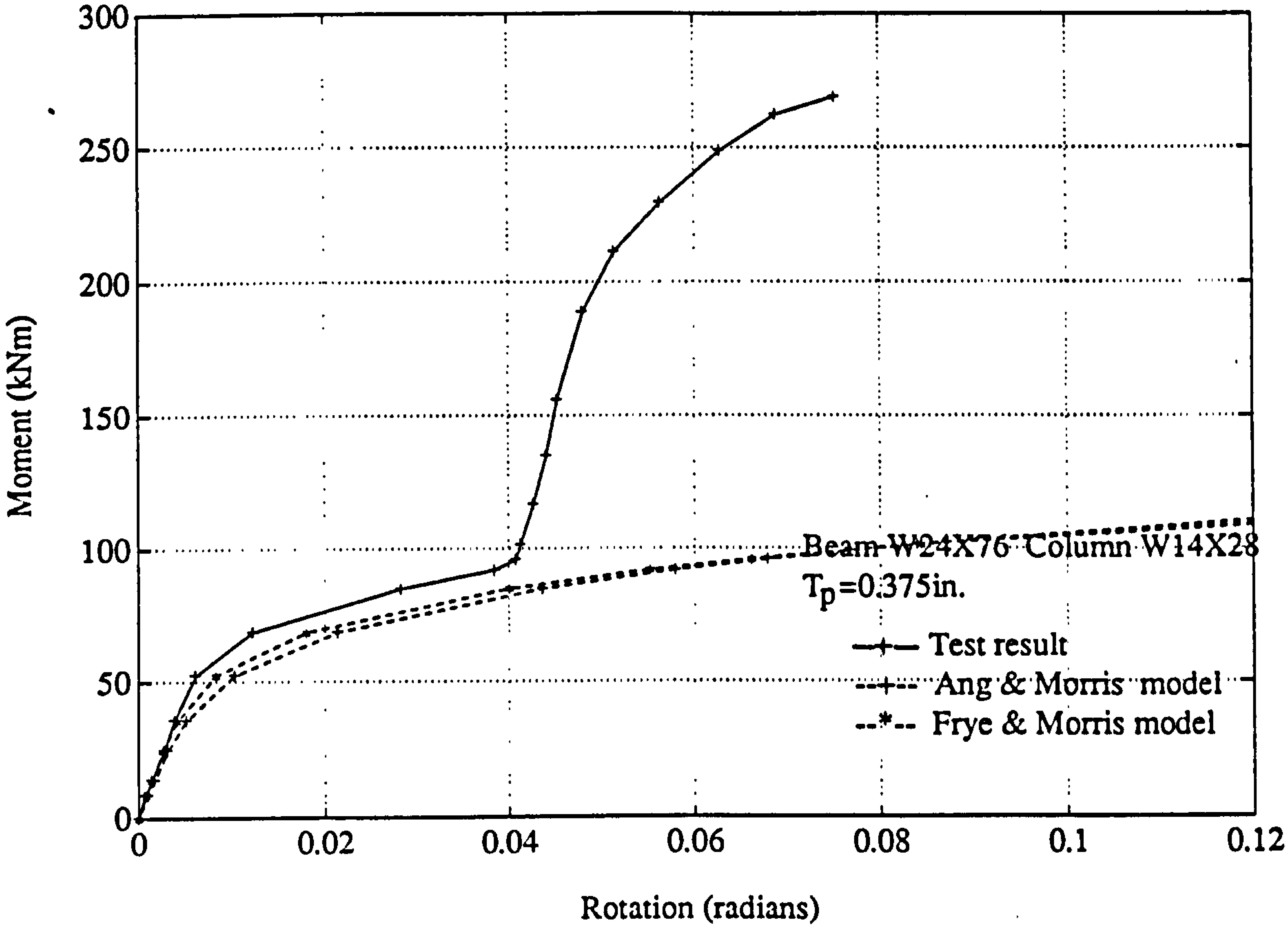


FIG.2-25 Comparison between analytical moment-rotation relationships and Sommer's test 16

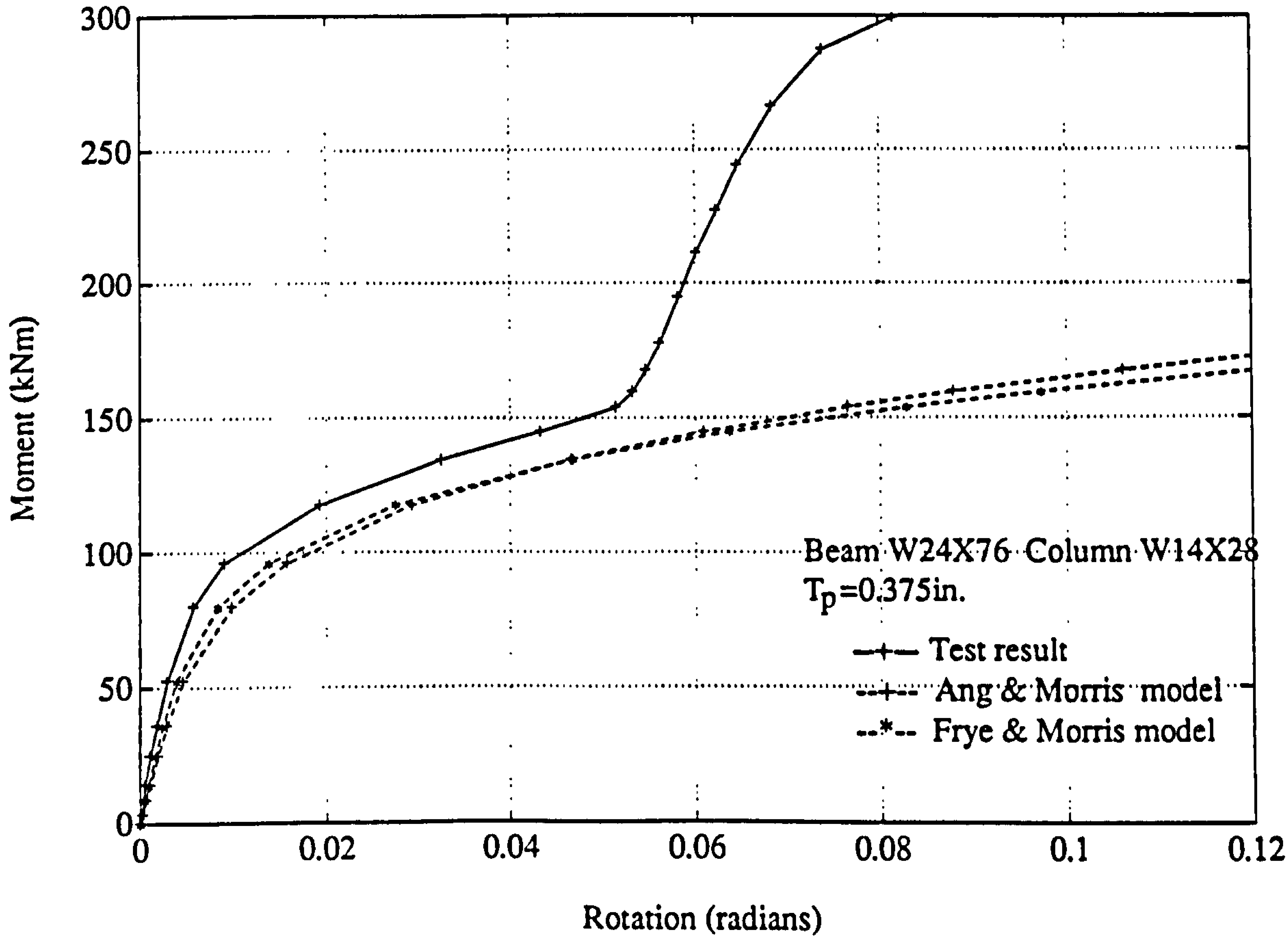


FIG.2-26 Comparison between analytical moment-rotation relationships and Sommer's test 17

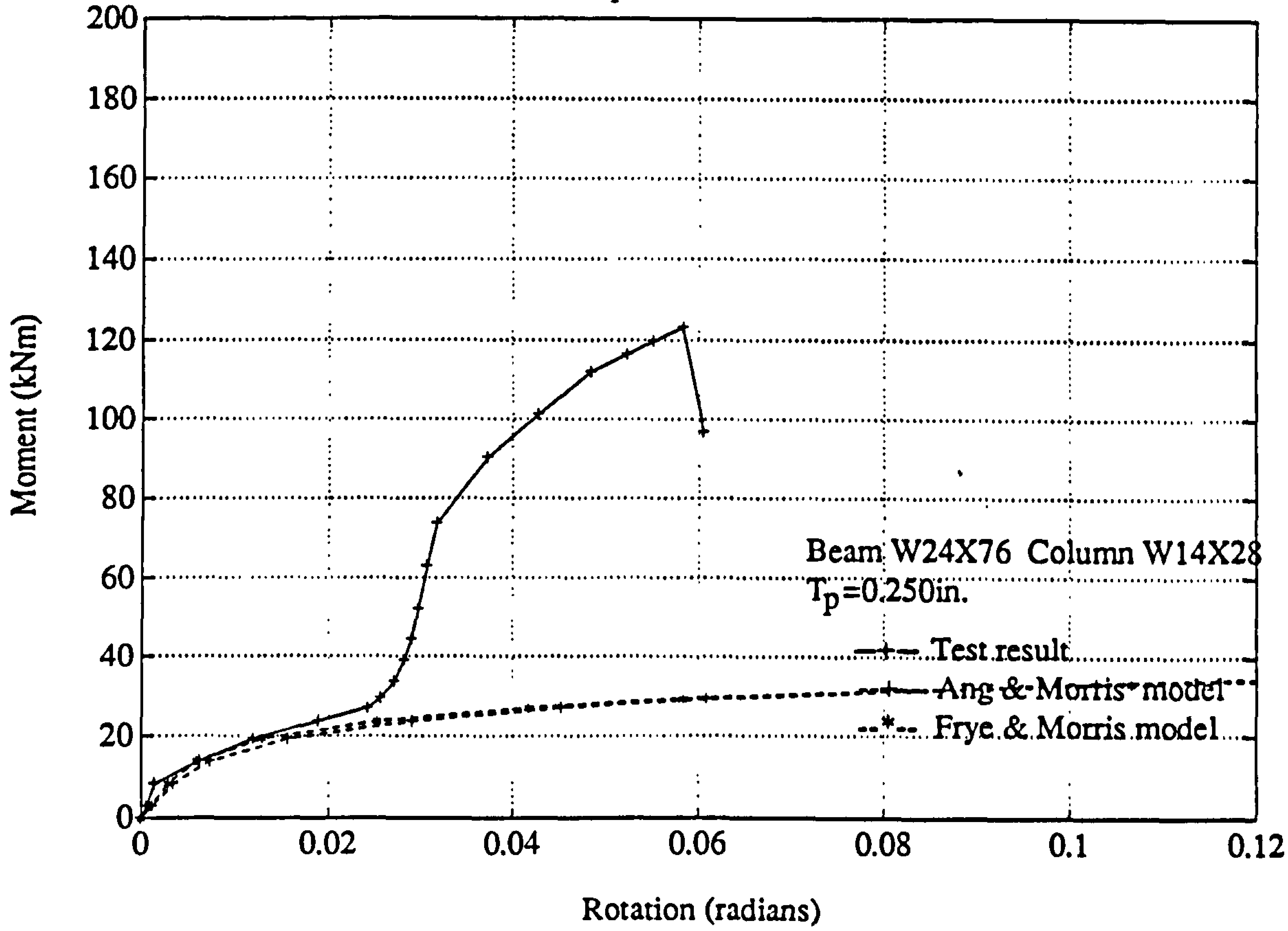


FIG.2-27 Comparison between analytical moment-rotation relationships and Sommer's test 18

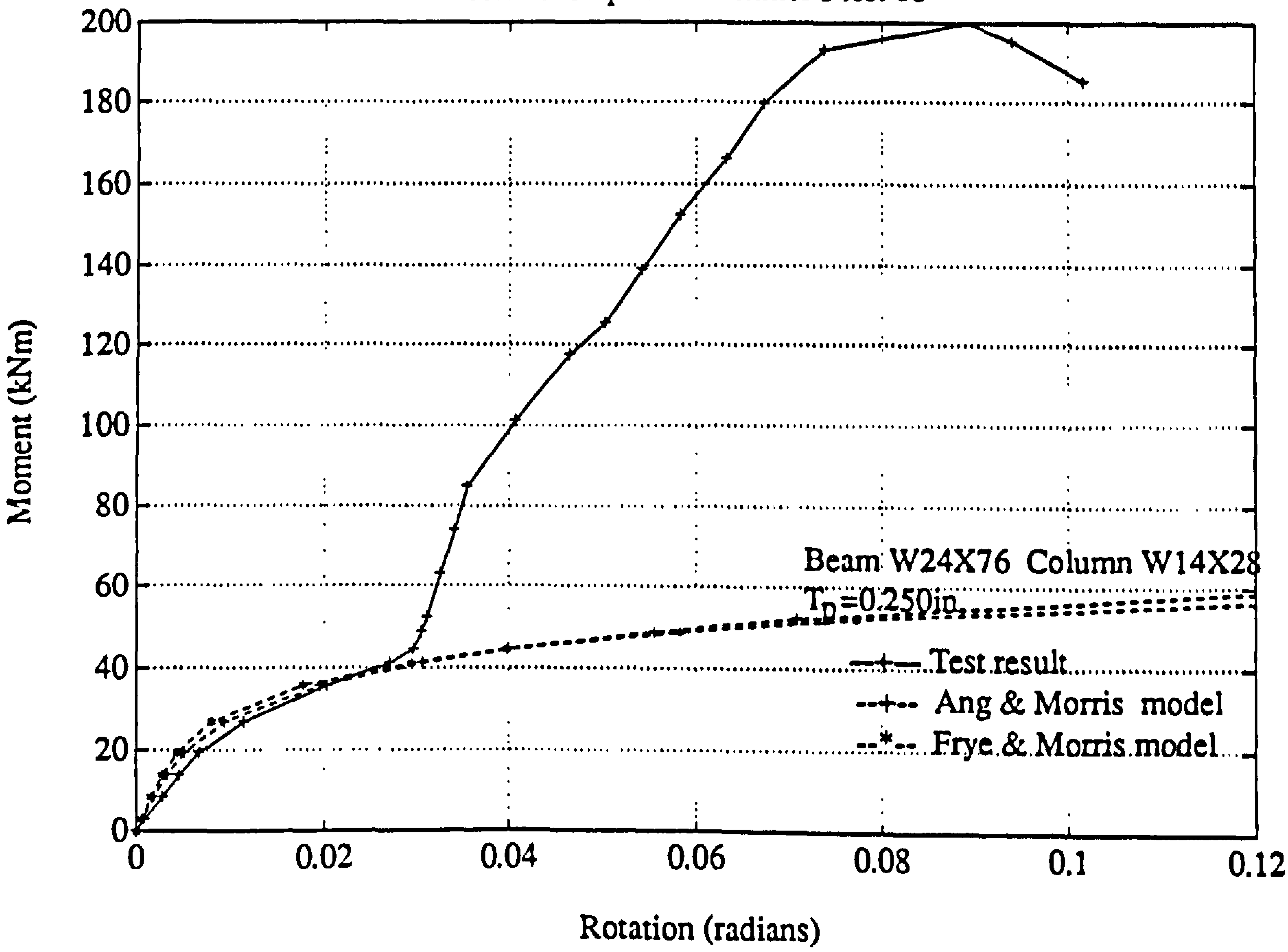


FIG.2-28 Comparison between analytical moment-rotation relationships and Sommer's test 19

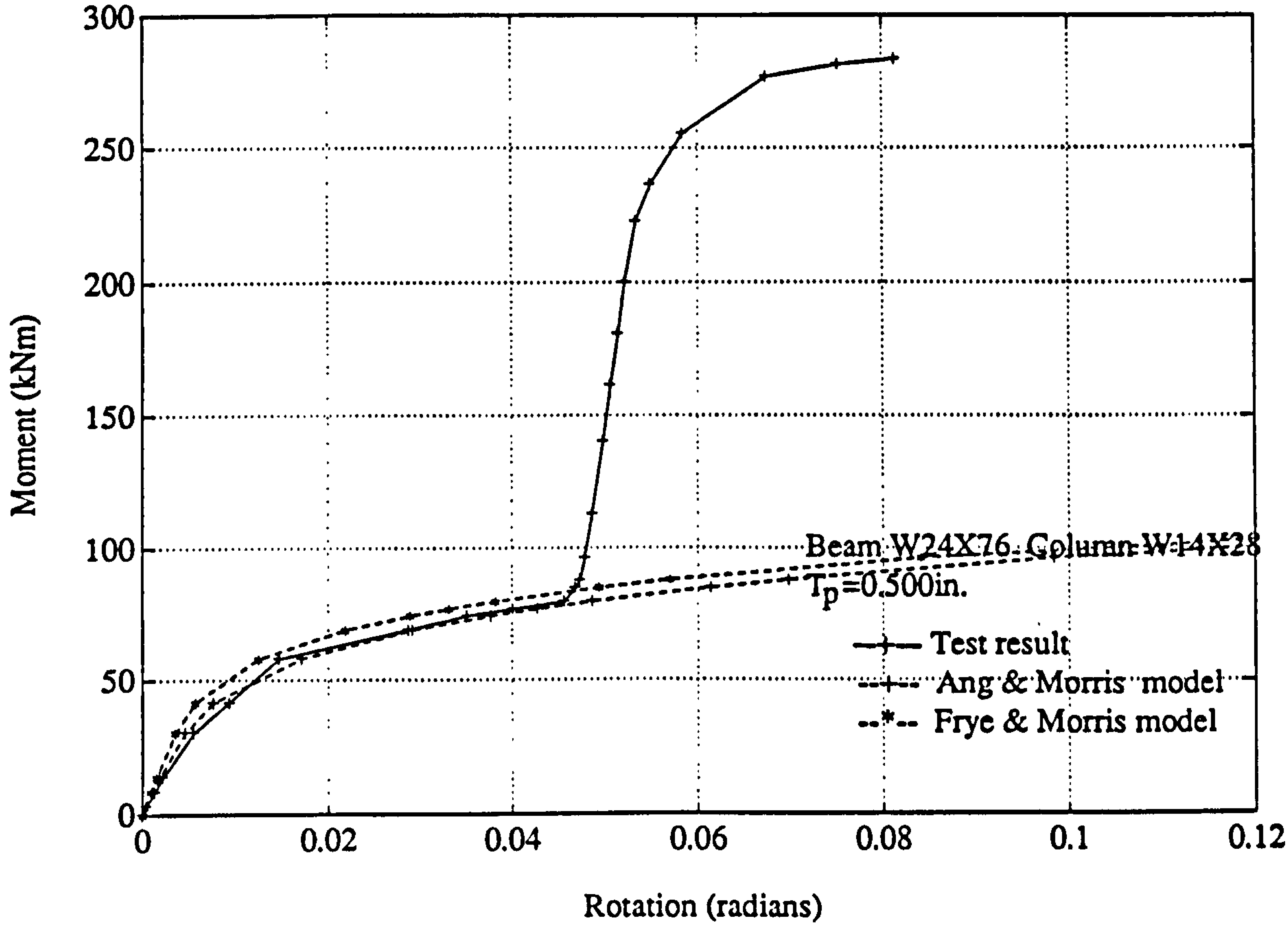


FIG.2-29 Comparison between analytical moment-rotation relationships and Sommer's test 20

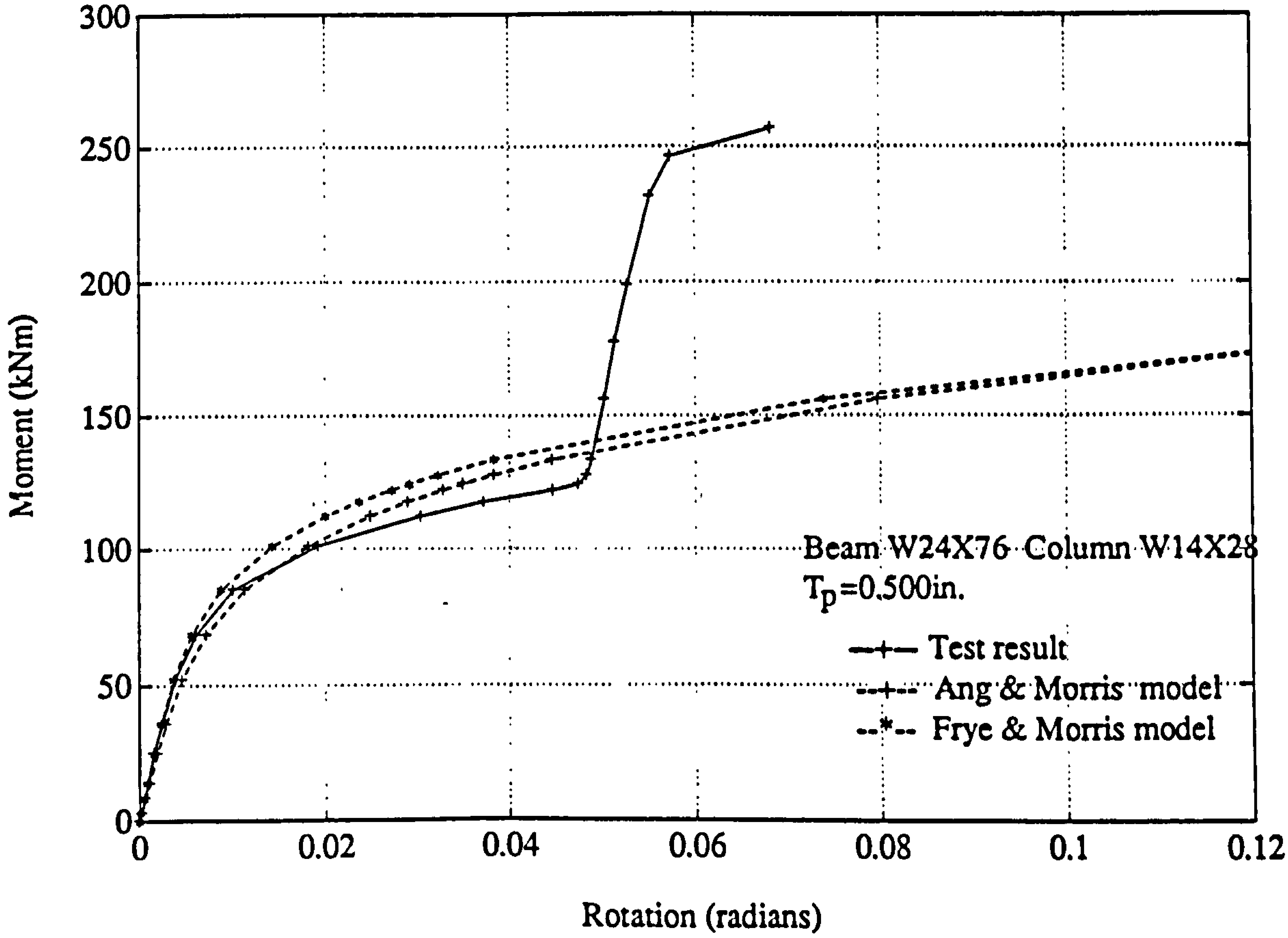


FIG.2-30 Comparison between analytical moment-rotation relationships and Sommer's test 25

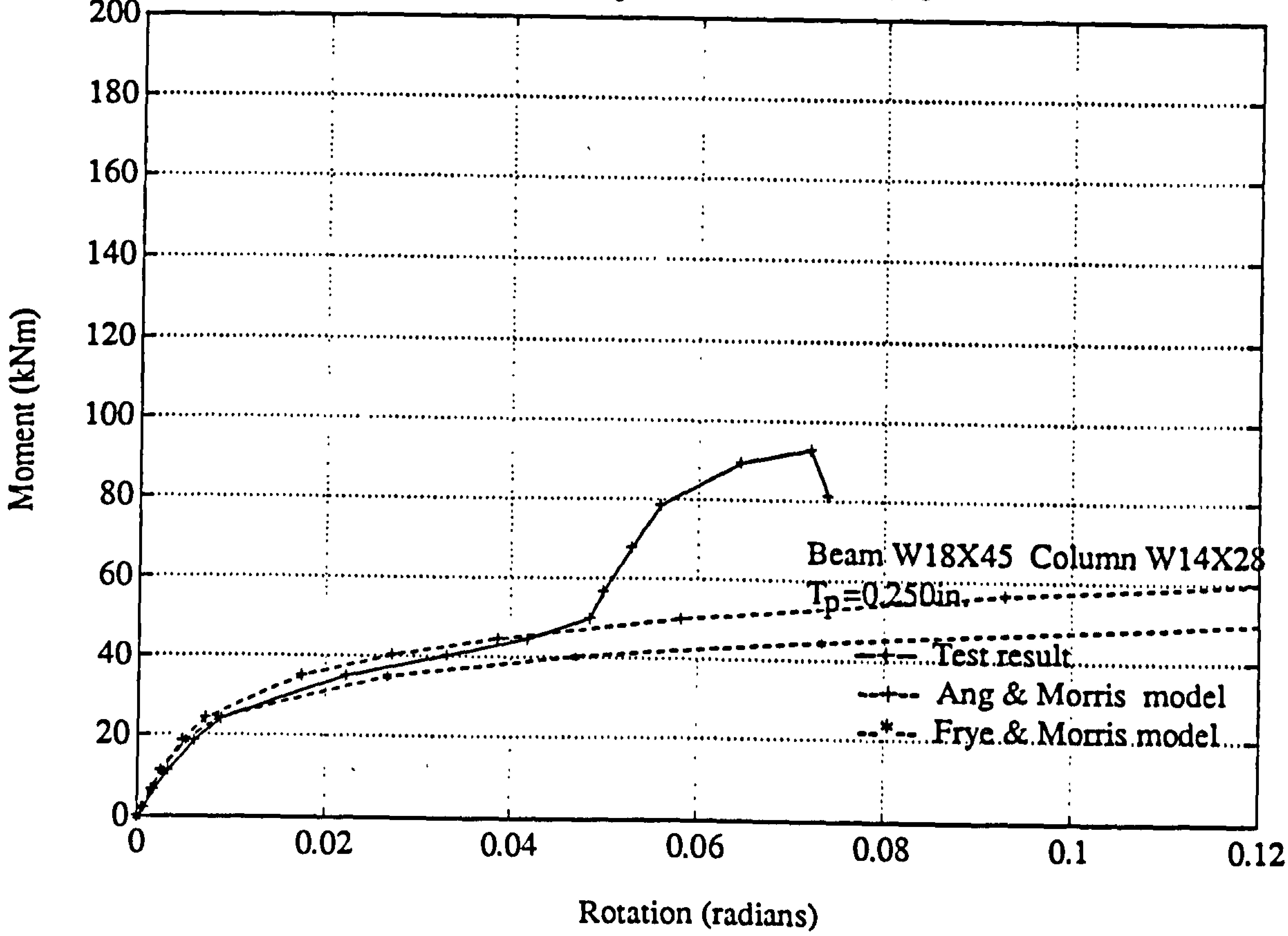


FIG.2-31 Comparison between analytical moment-rotation relationships and Sommer's test 26

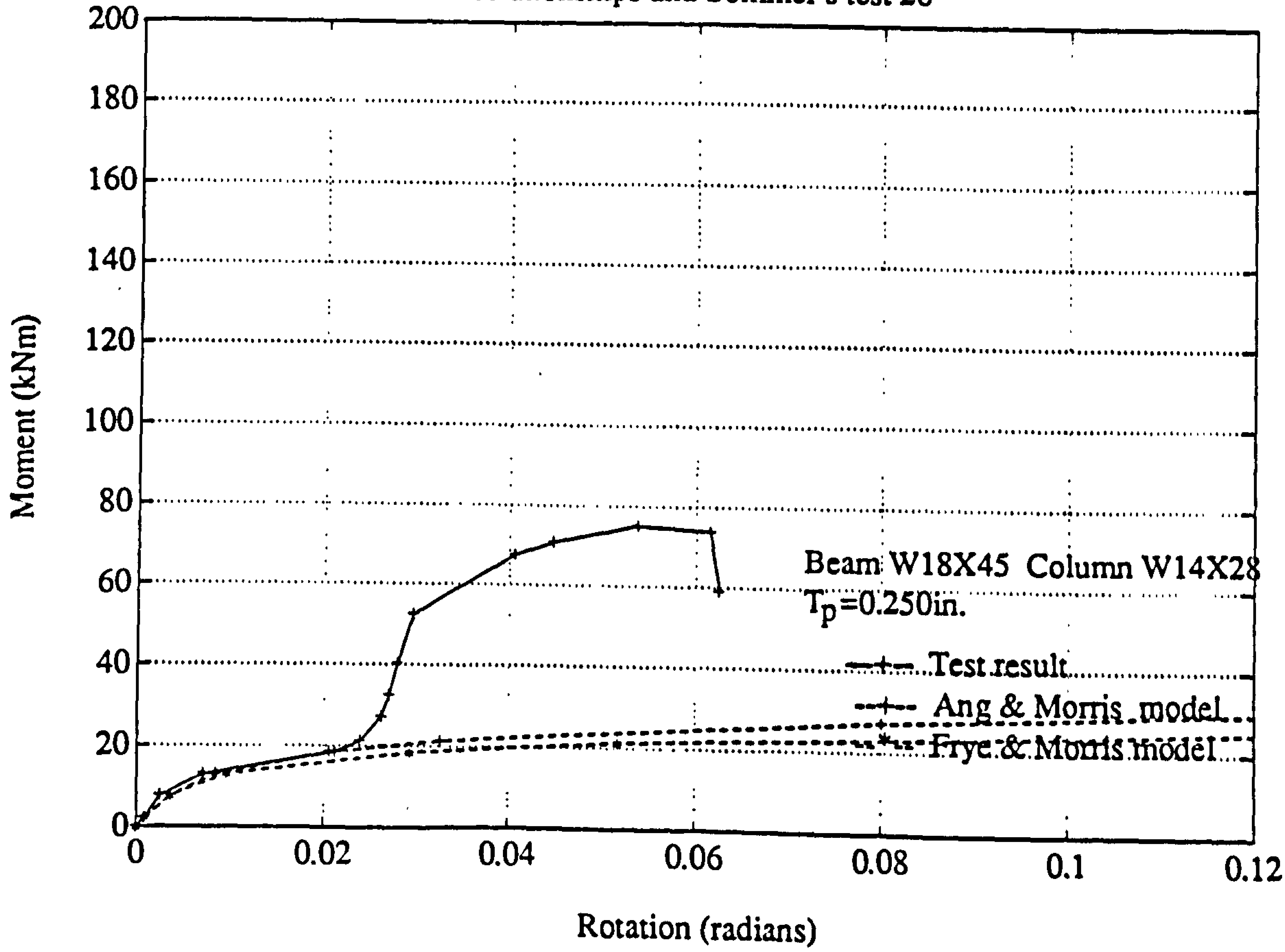


FIG.2-32 Comparison between analytical moment-rotation relationships and Sommer's test 27

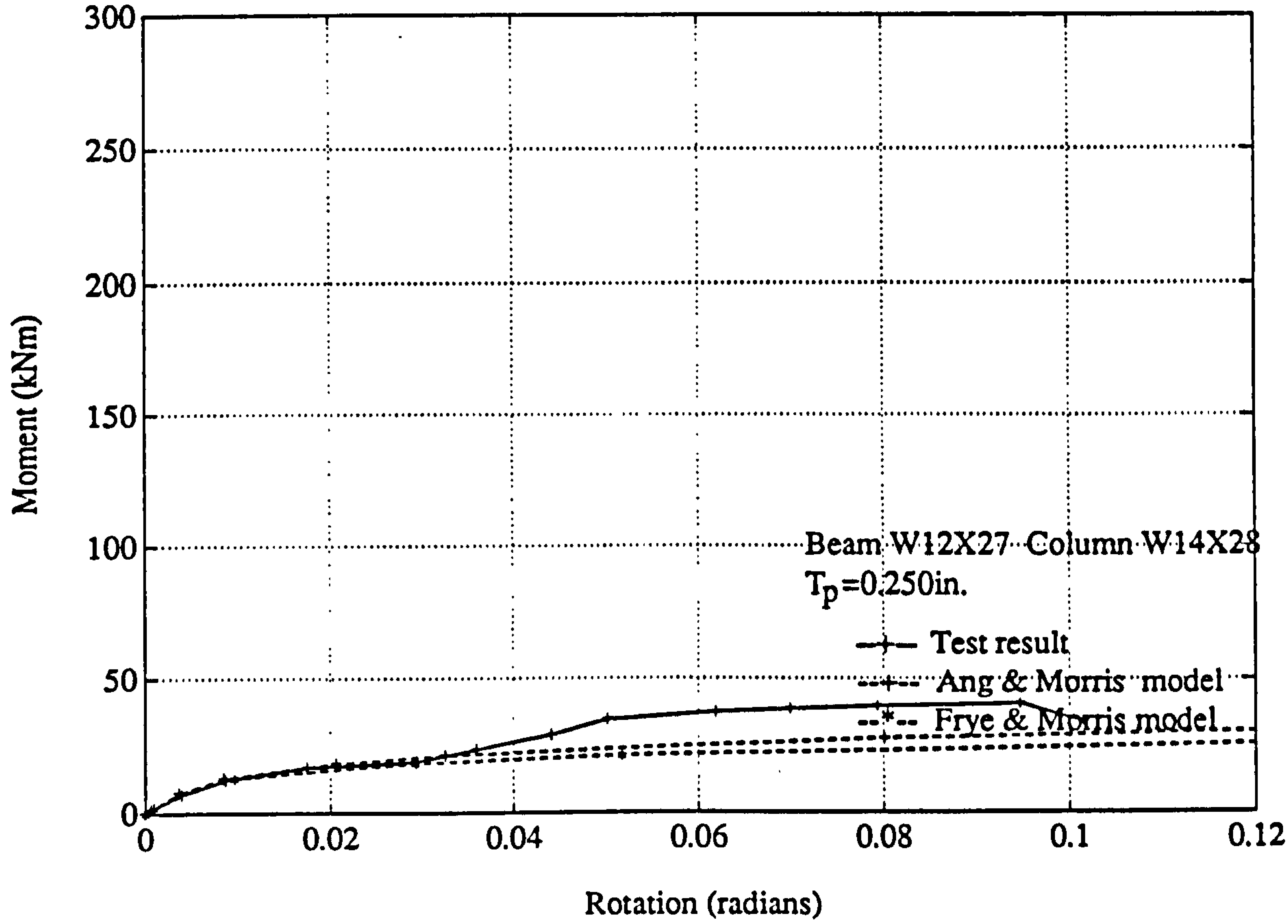


FIG.2-33 Comparison between analytical moment-rotation relationships and Sommer's test 28

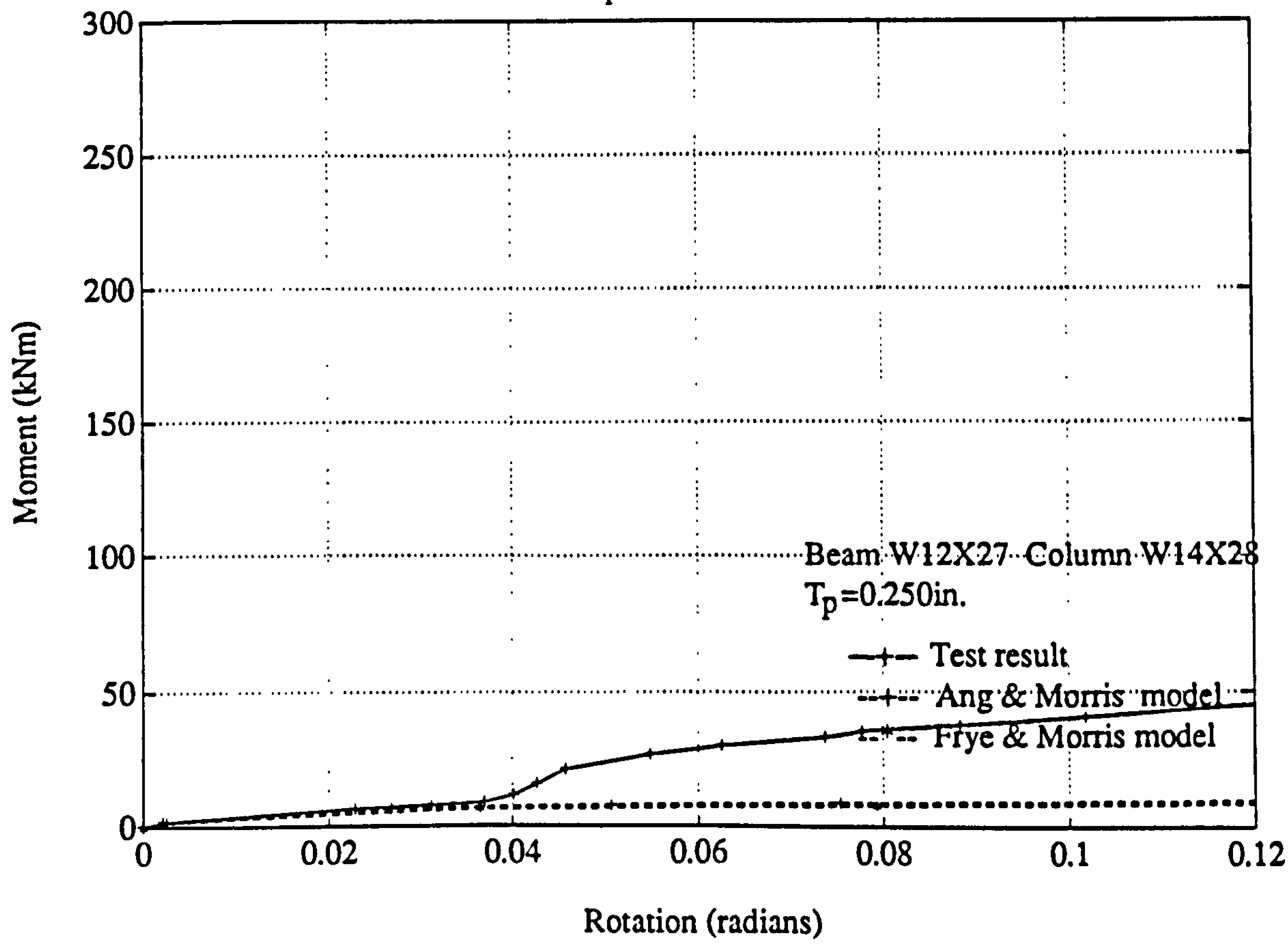


FIG.2-34 Comparison between analytical moment-rotation relationships and Hafez's test 1

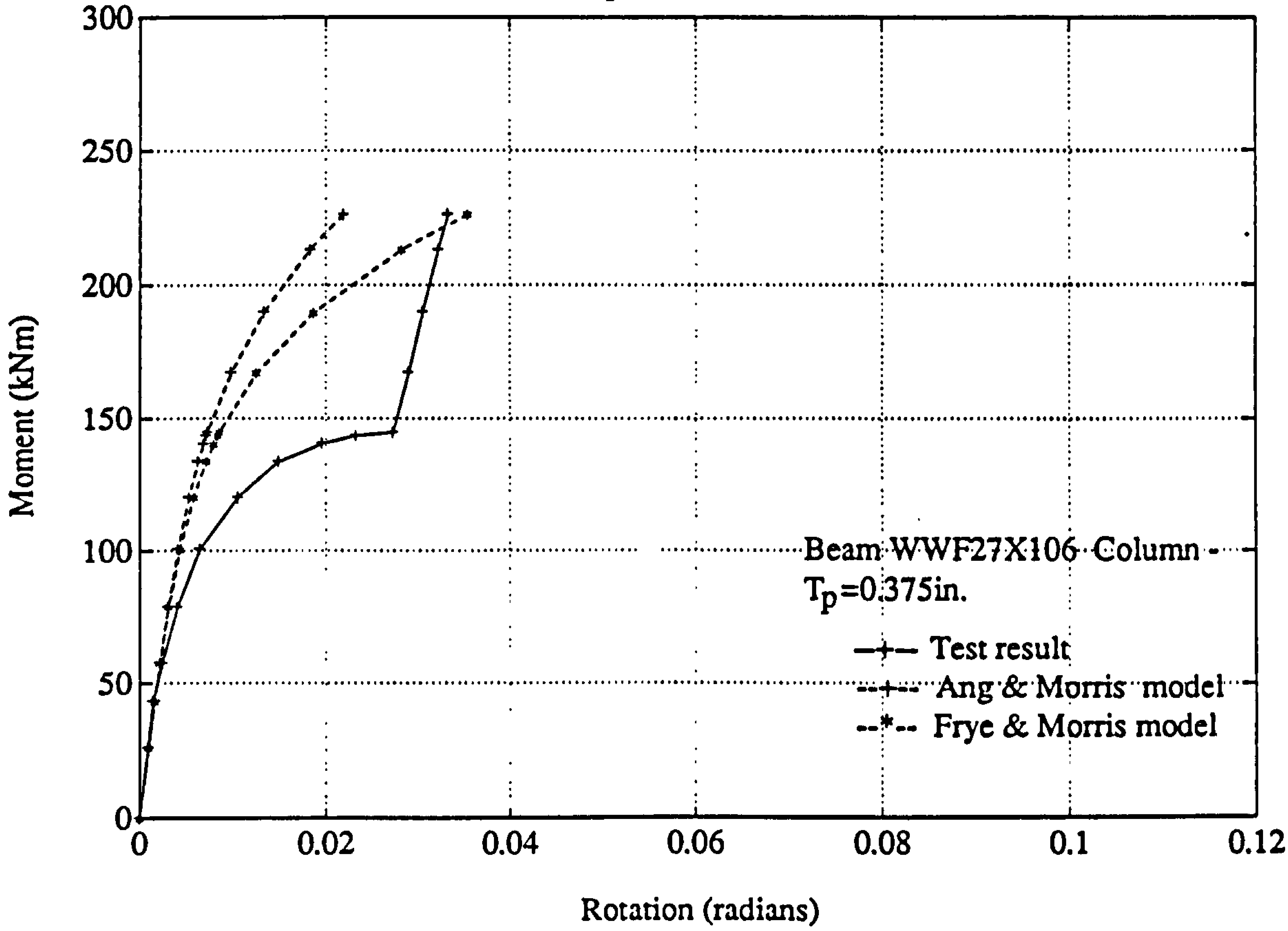


FIG.2-35 Comparison between analytical moment-rotation relationships and Hafez's test 2

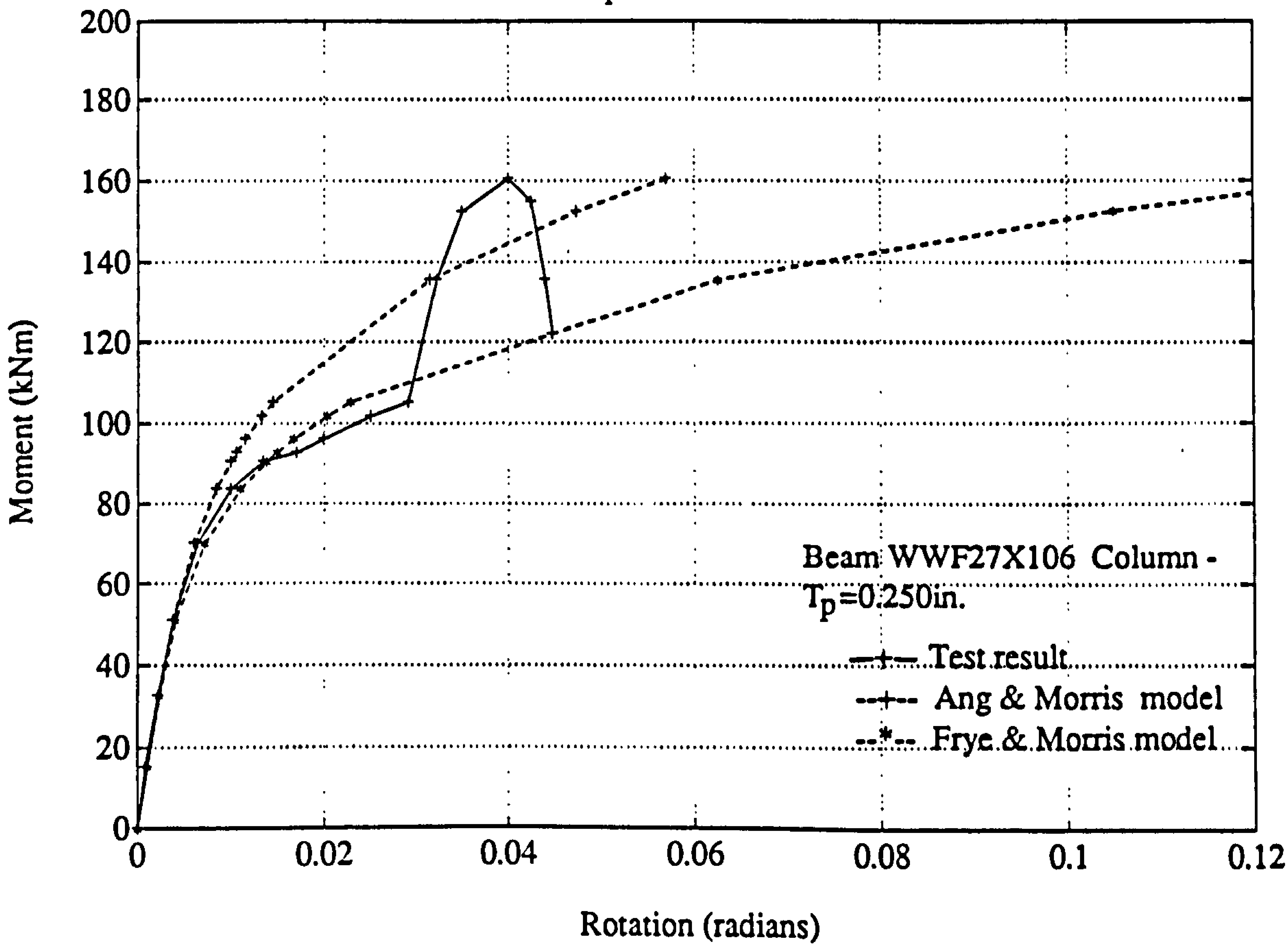


FIG.2-36 Comparison between analytical moment-rotation relationships and Hafez's test 3

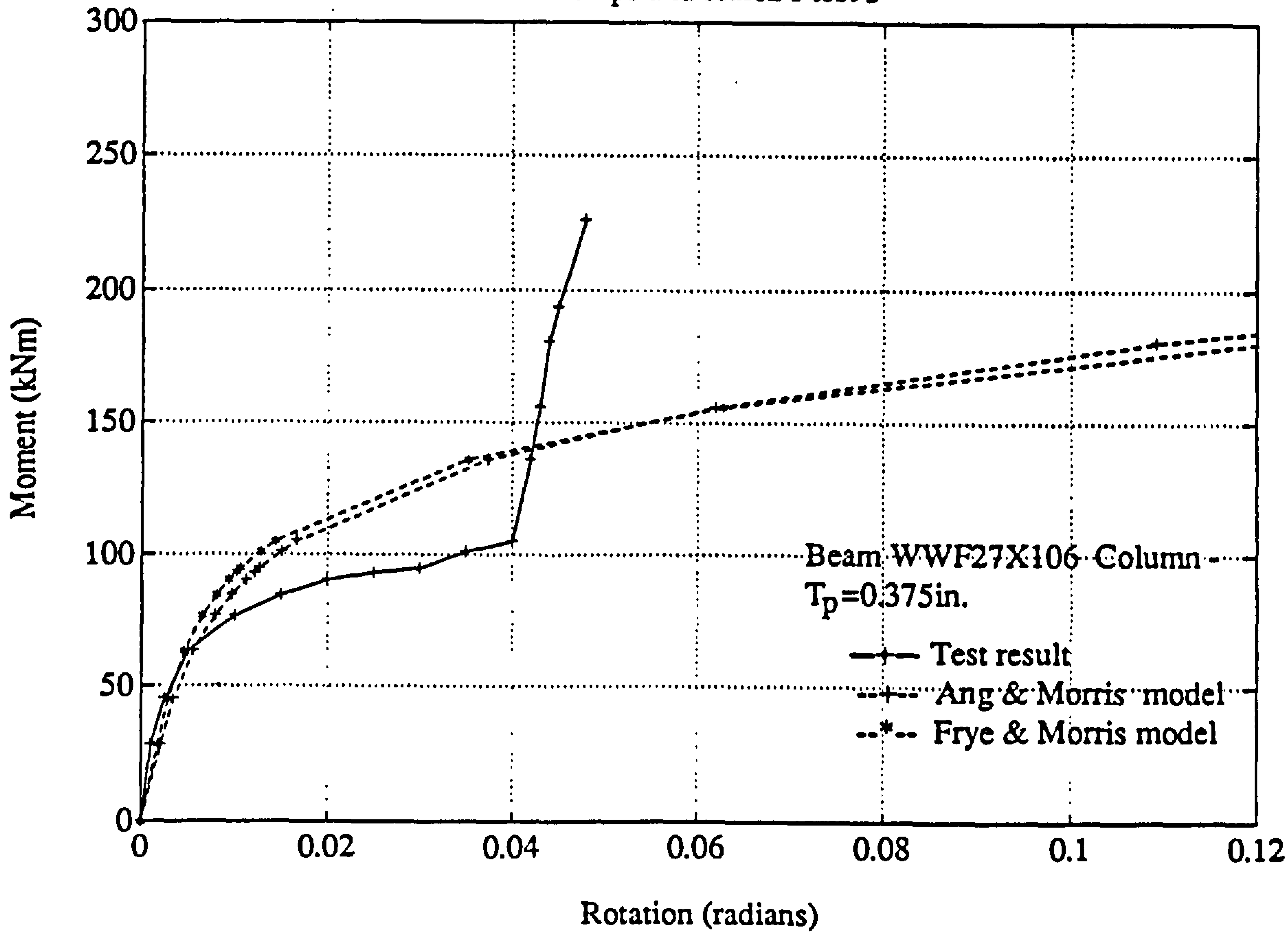


FIG.2-37 Comparison between analytical moment-rotation relationships and Hafez's test 4

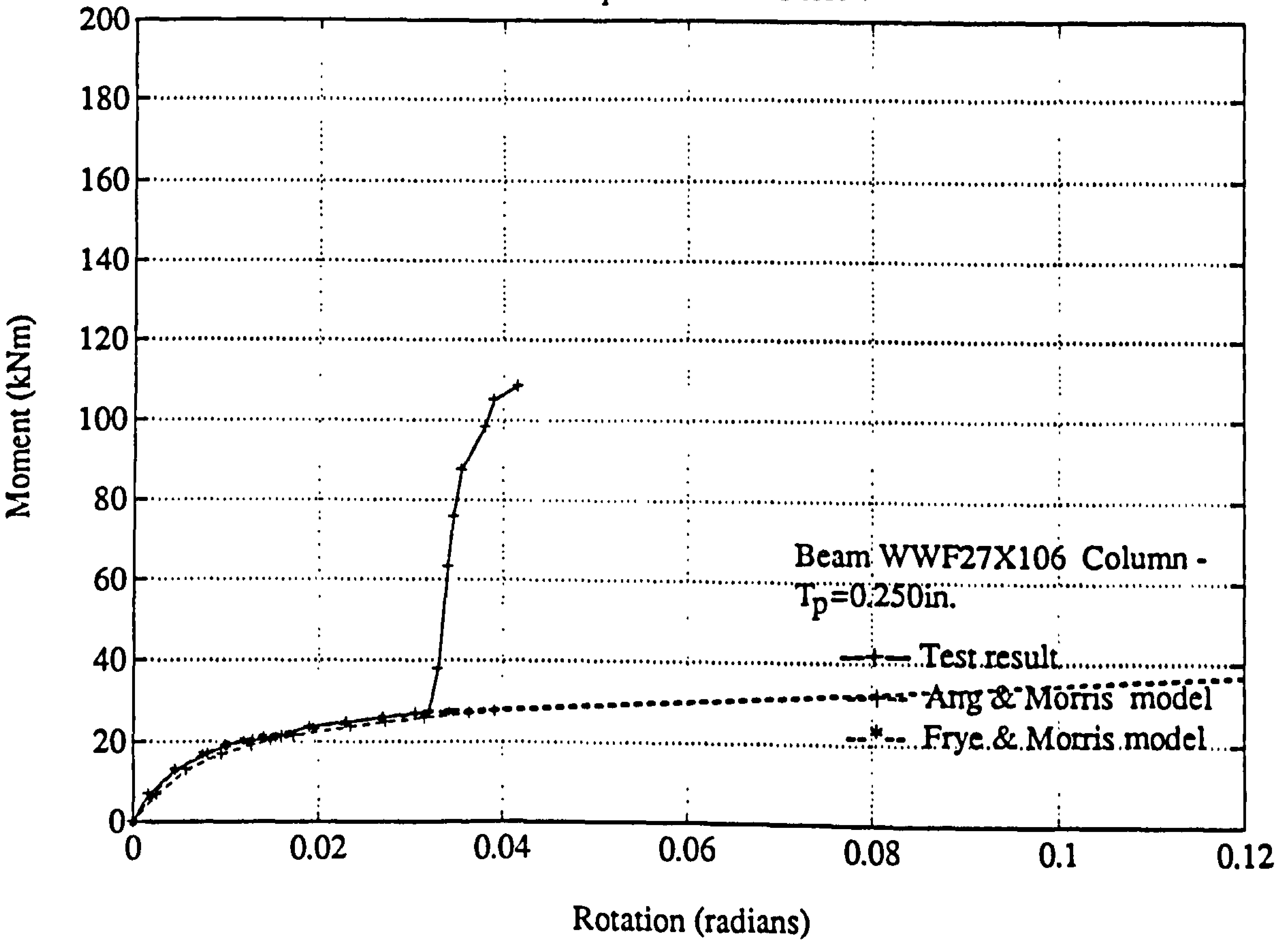


FIG.2-38 Comparison between analytical moment-rotation relationships and Hafez's test 5

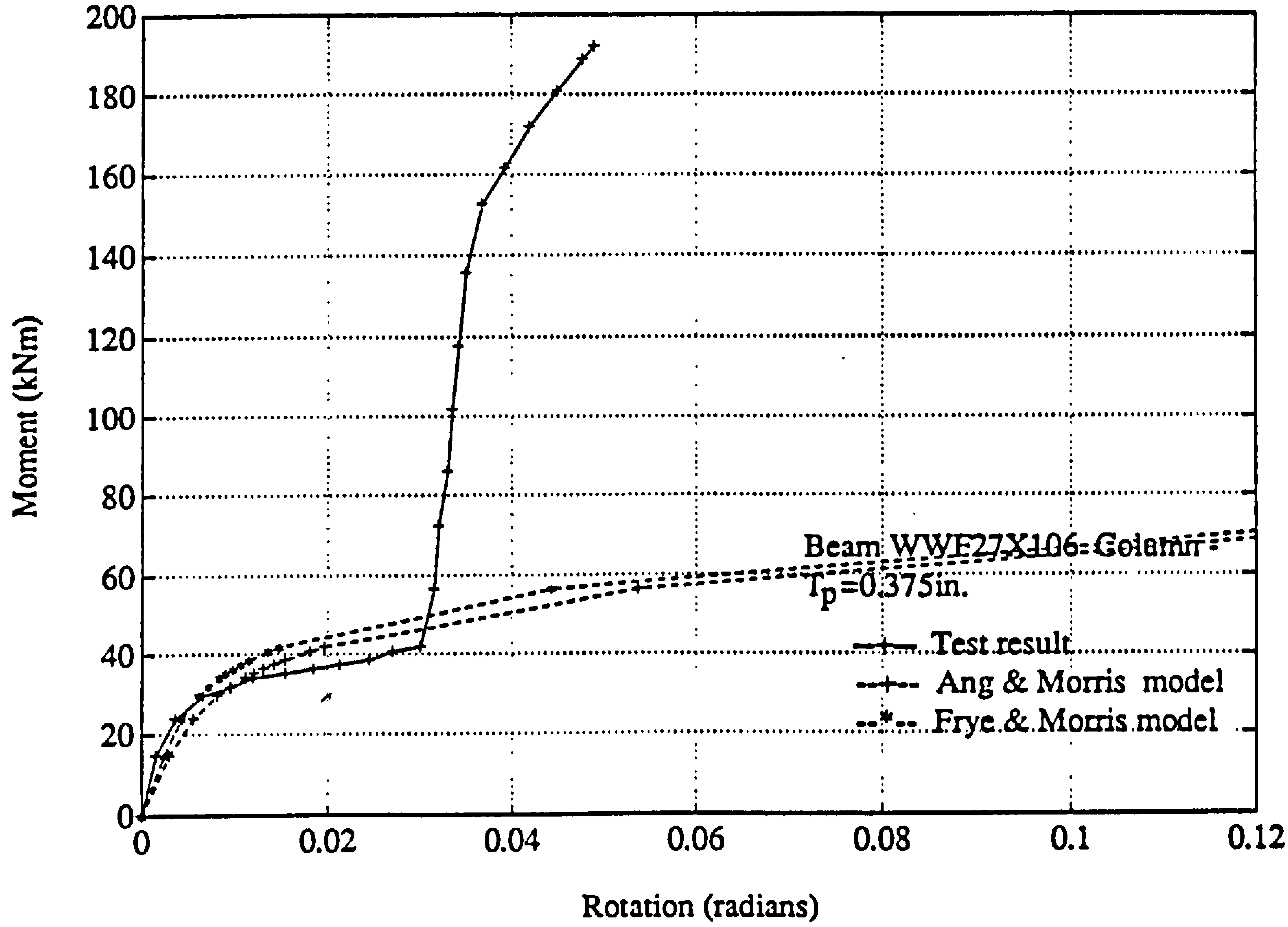


FIG.2-39 Comparison between analytical moment-rotation relationships and Hafez's test 6

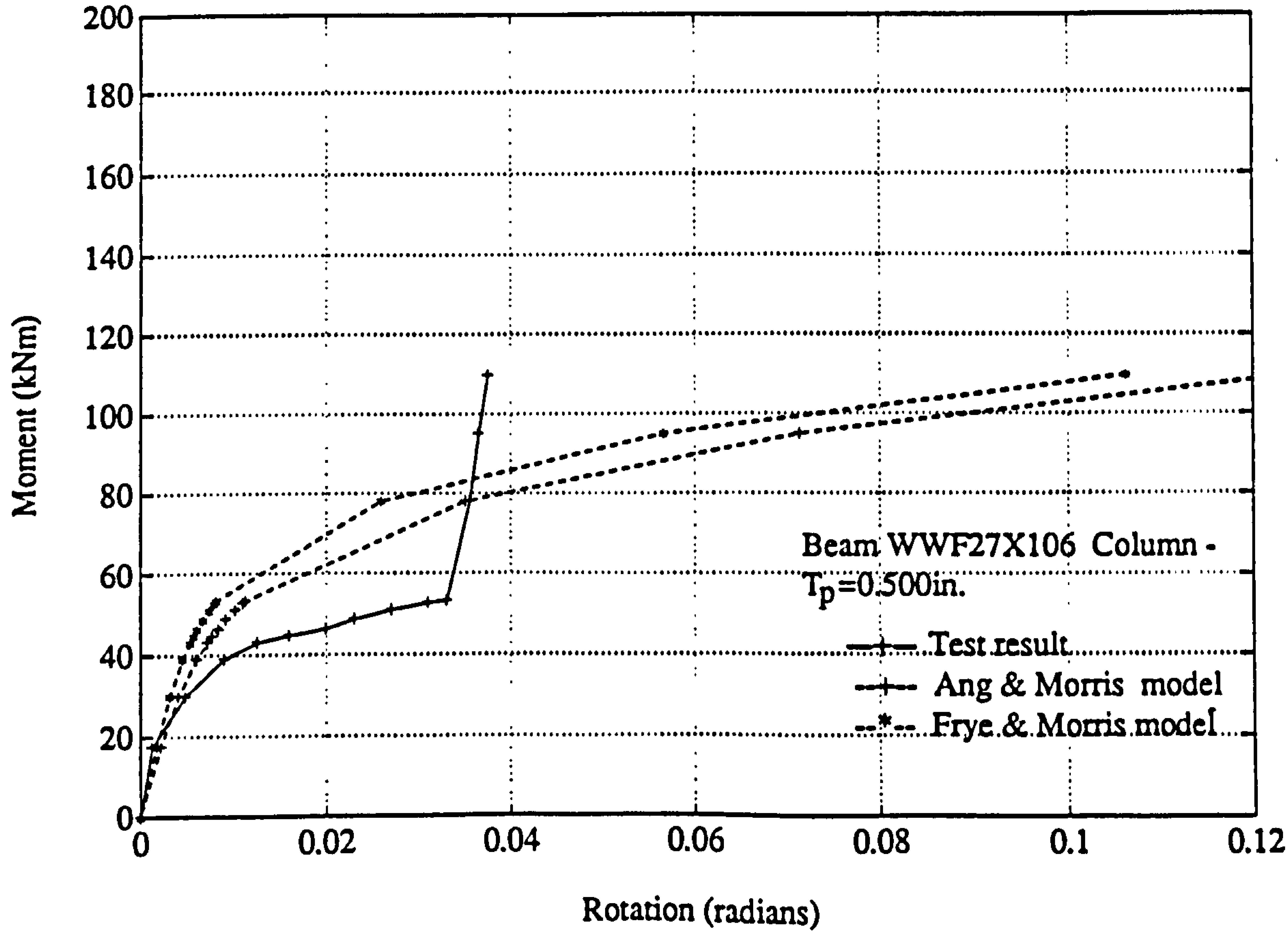


FIG.2-40 Comparison between analytical moment-rotation relationships and Hafez's test 7

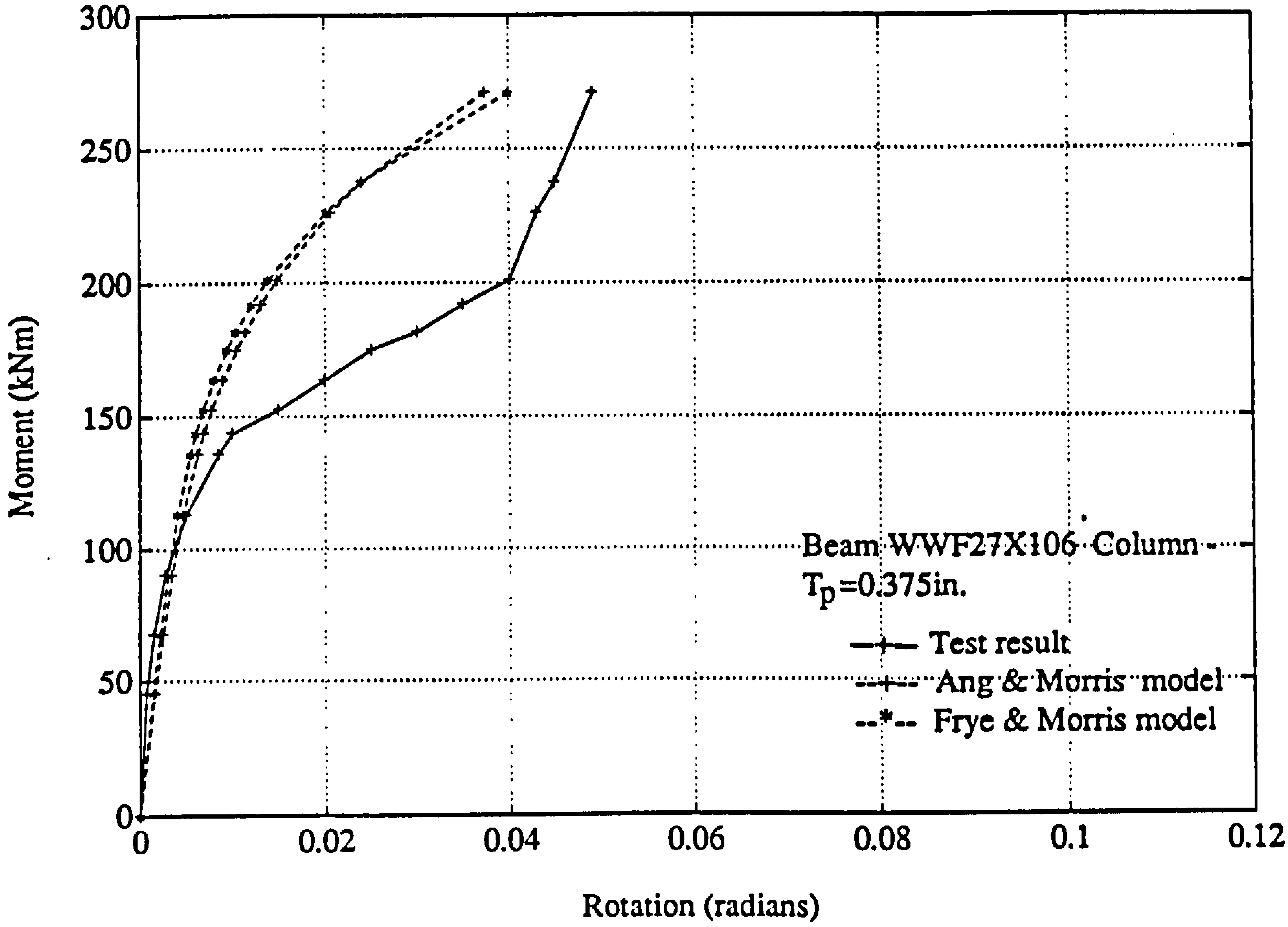


FIG.2-41 Comparison between analytical moment-rotation relationships and Hafez's test 8

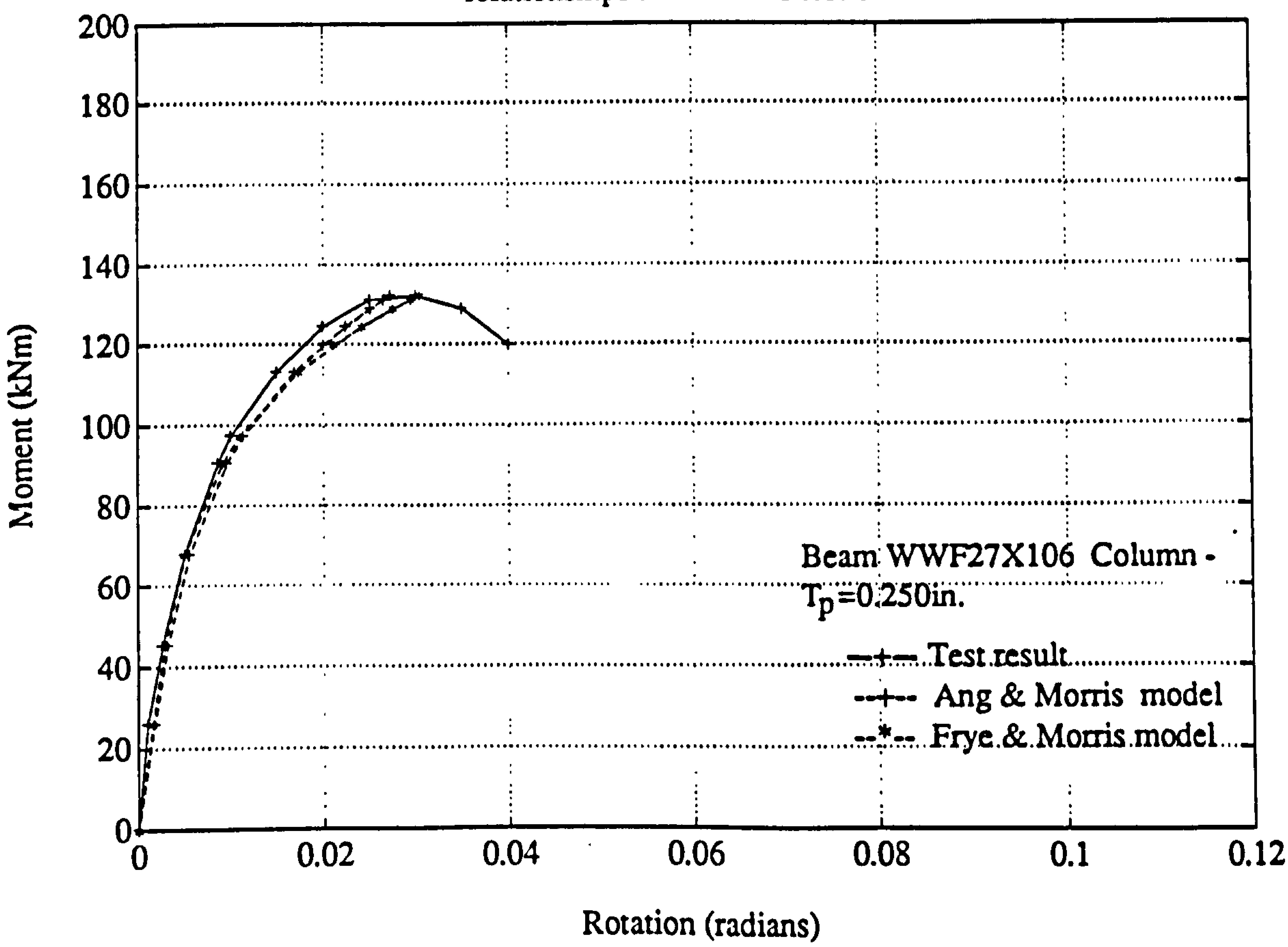


FIG.2-42 Comparison between analytical moment-rotation relationships and Zandonini & Zanon test HP1-1

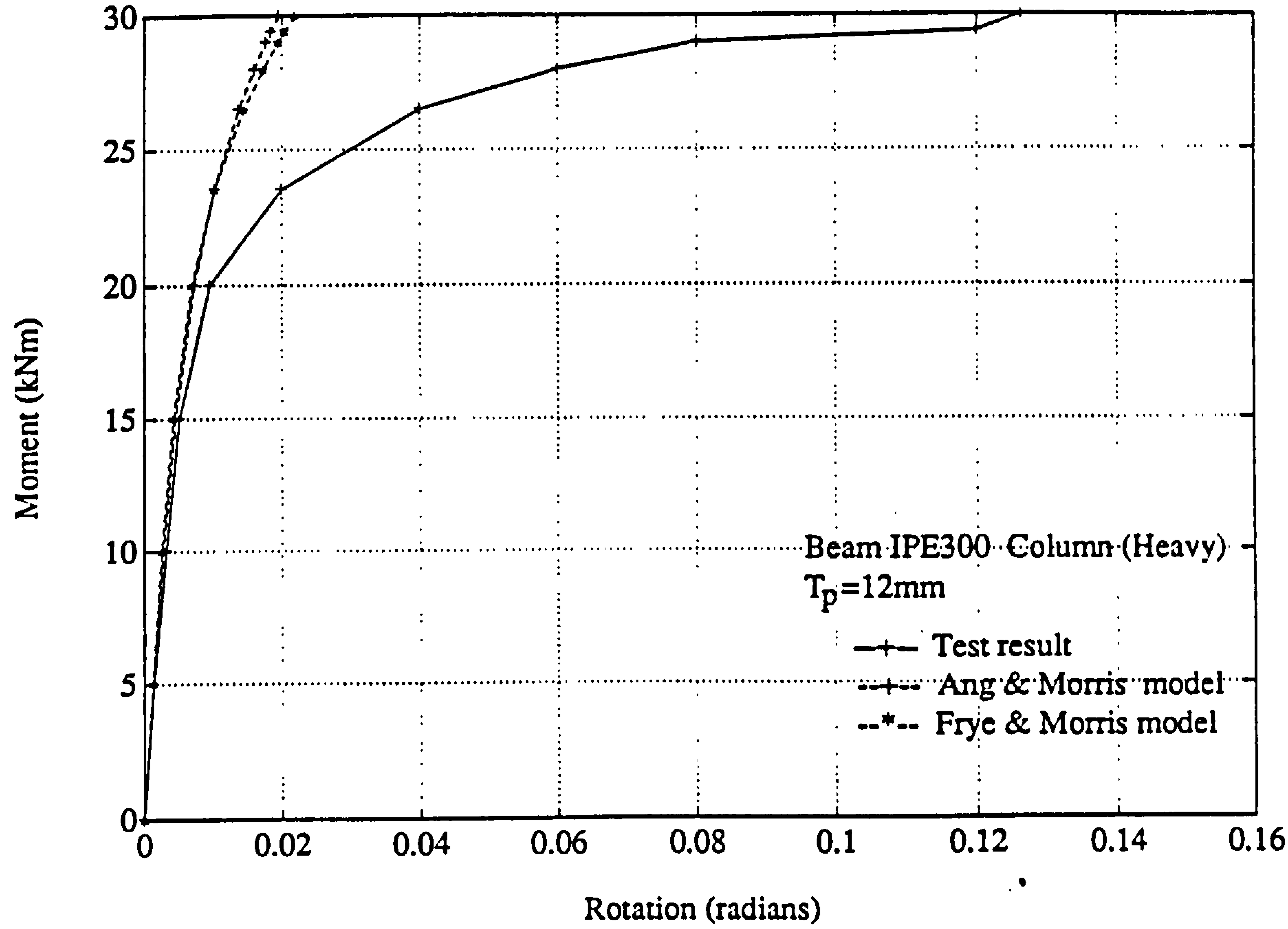


FIG.2-43 Comparison between analytical moment-rotation relationships and Zandonini & Zanon test HP1-2

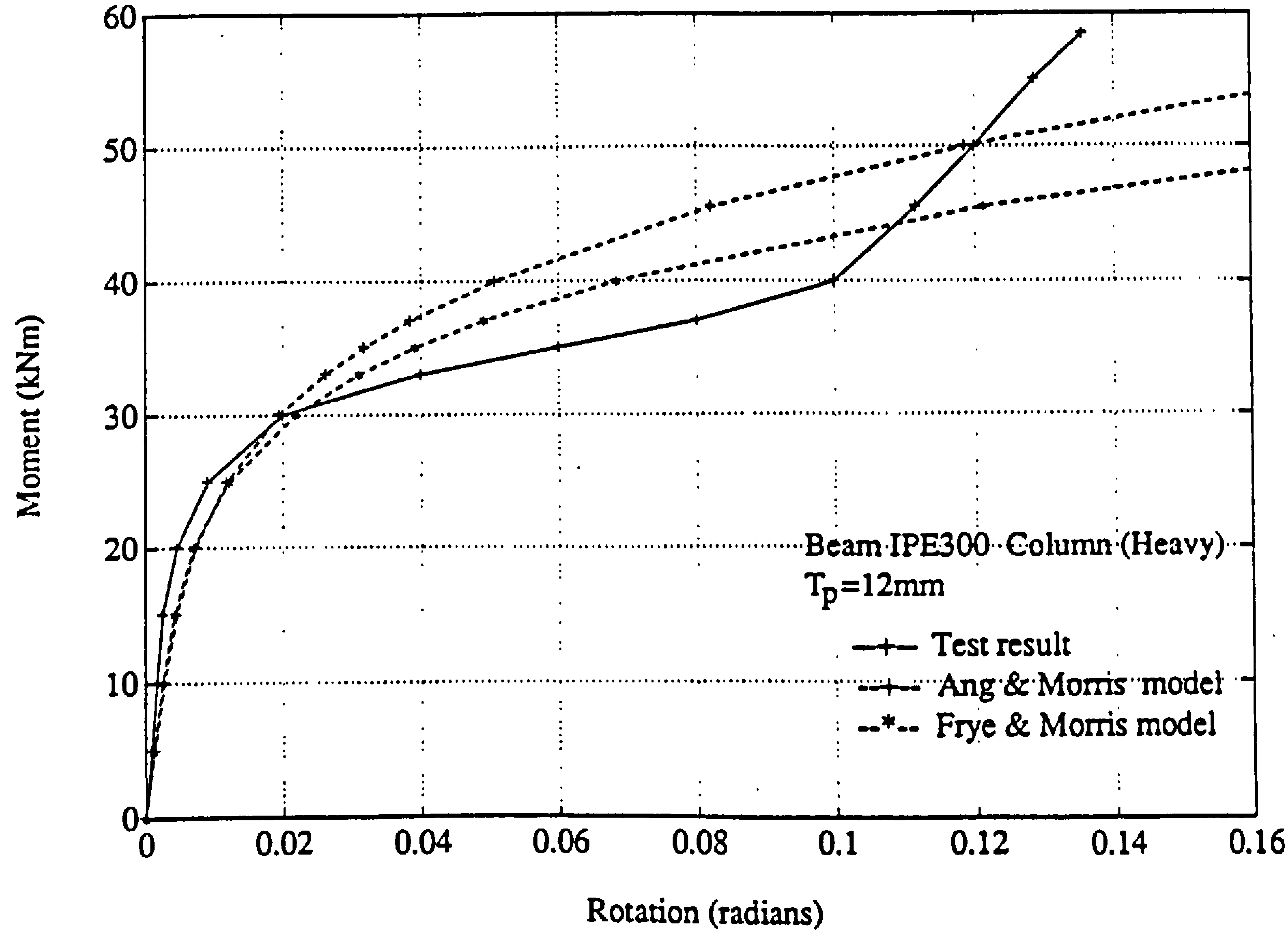


FIG.2-44 Comparison between analytical moment-rotation relationships and Zandonini & Zanon test HP1-3

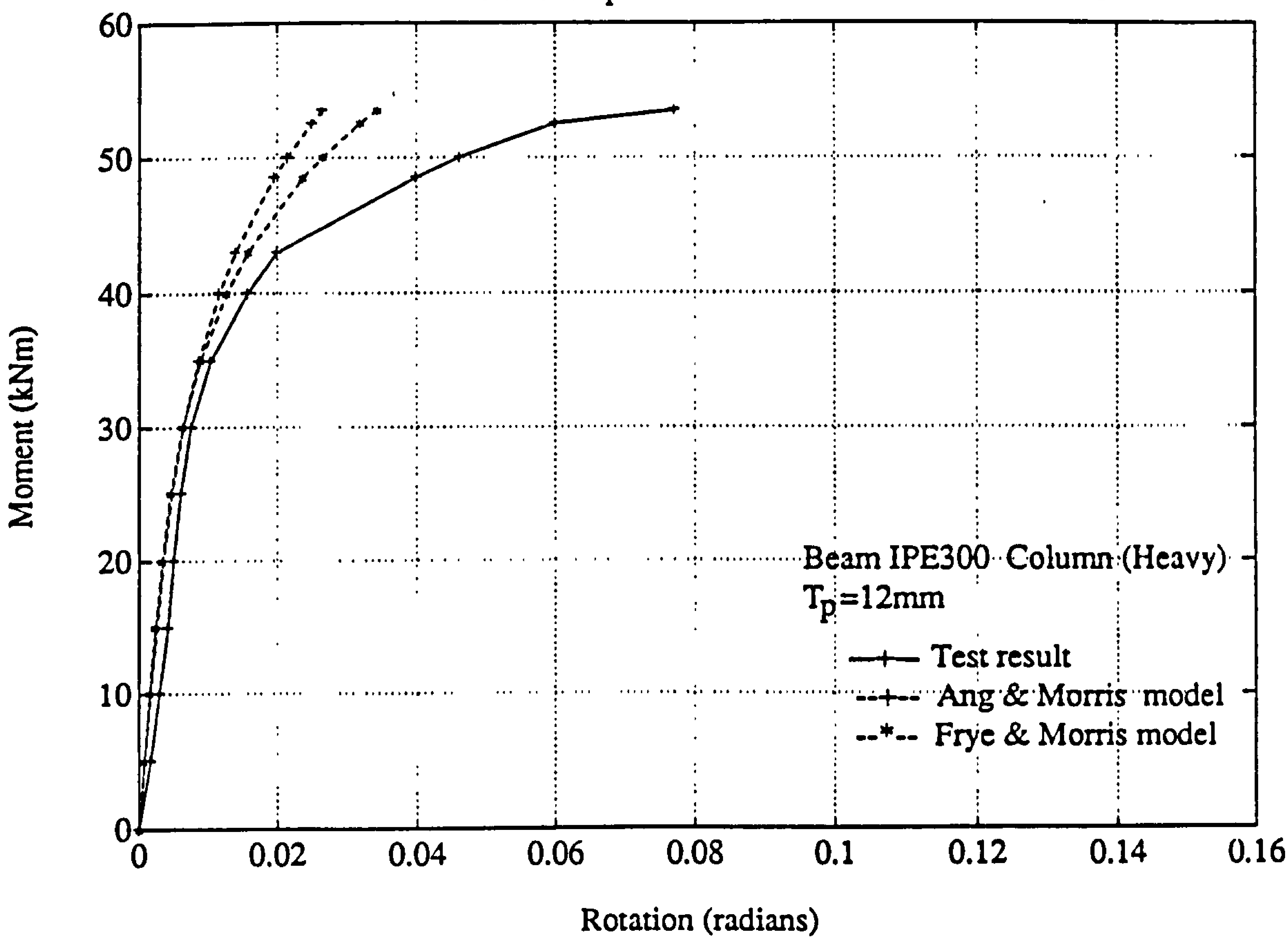


FIG.2-45 Comparison between analytical moment-rotation relationships and Van Dalen & MacIntyre

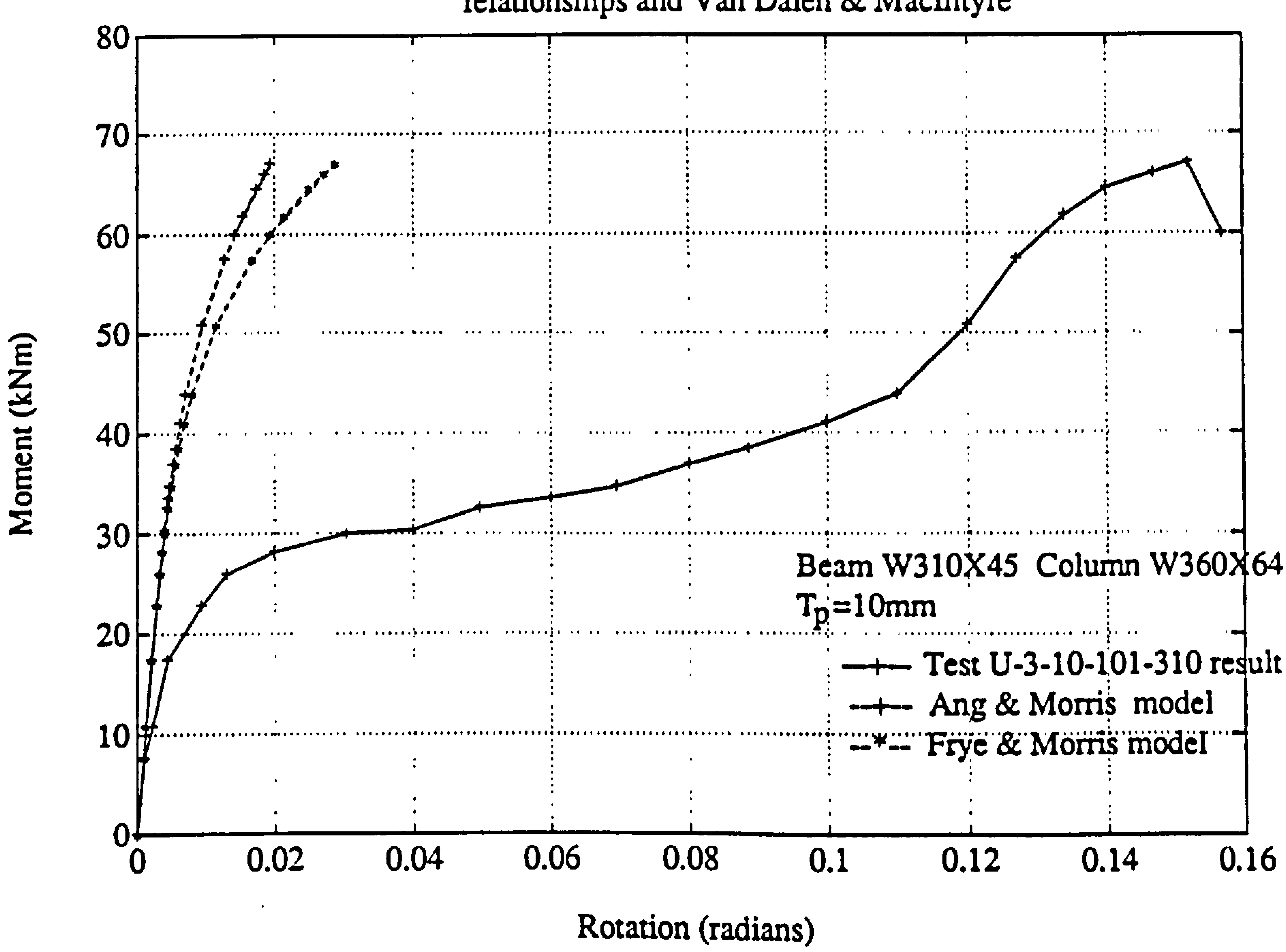


FIG.2-46 Comparison between analytical moment-rotation relationships and Van Dalen & MacIntyre

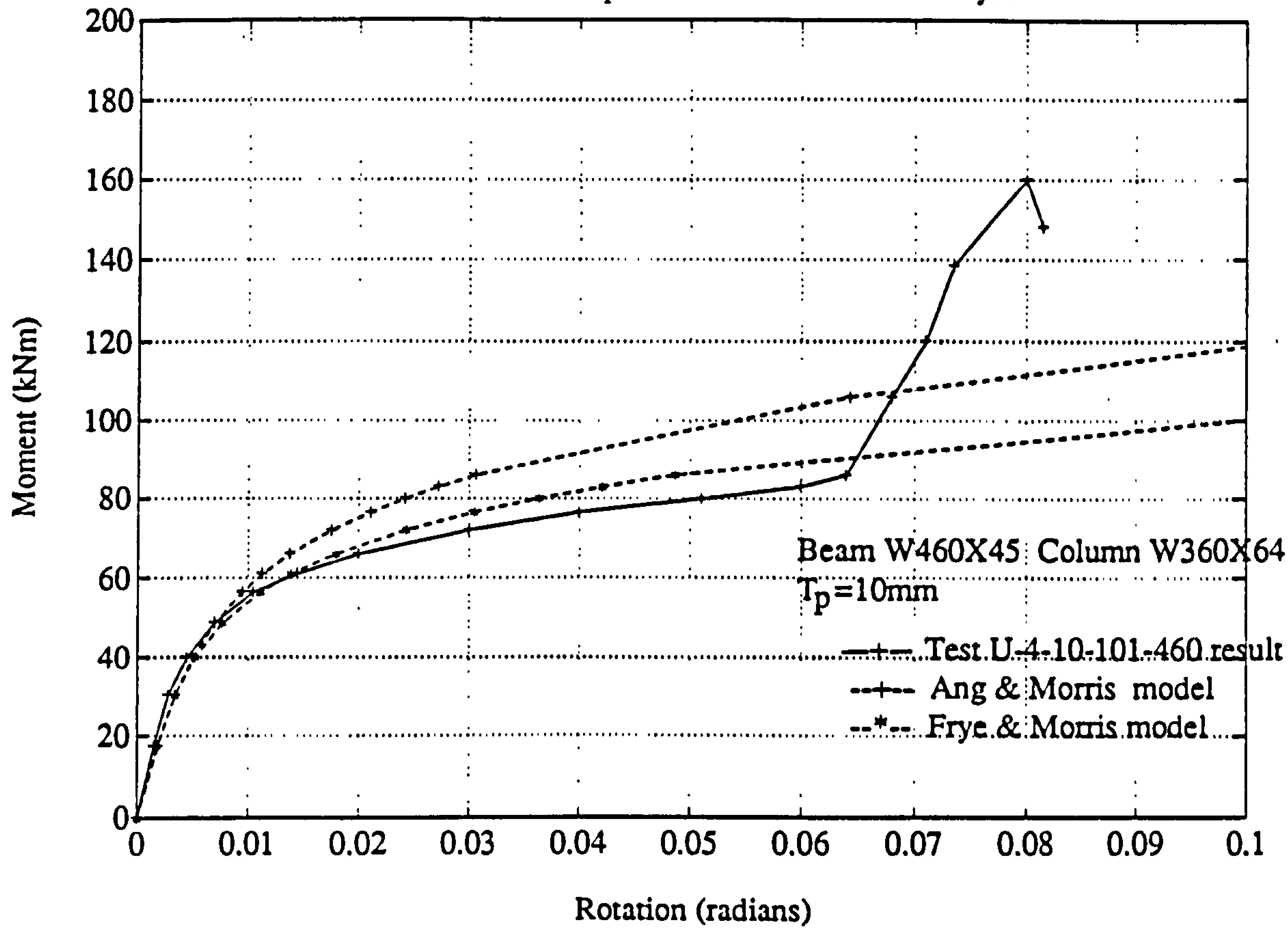


FIG.2-47 Comparison between analytical moment-rotation relationships and Van Dalen & MacIntyre

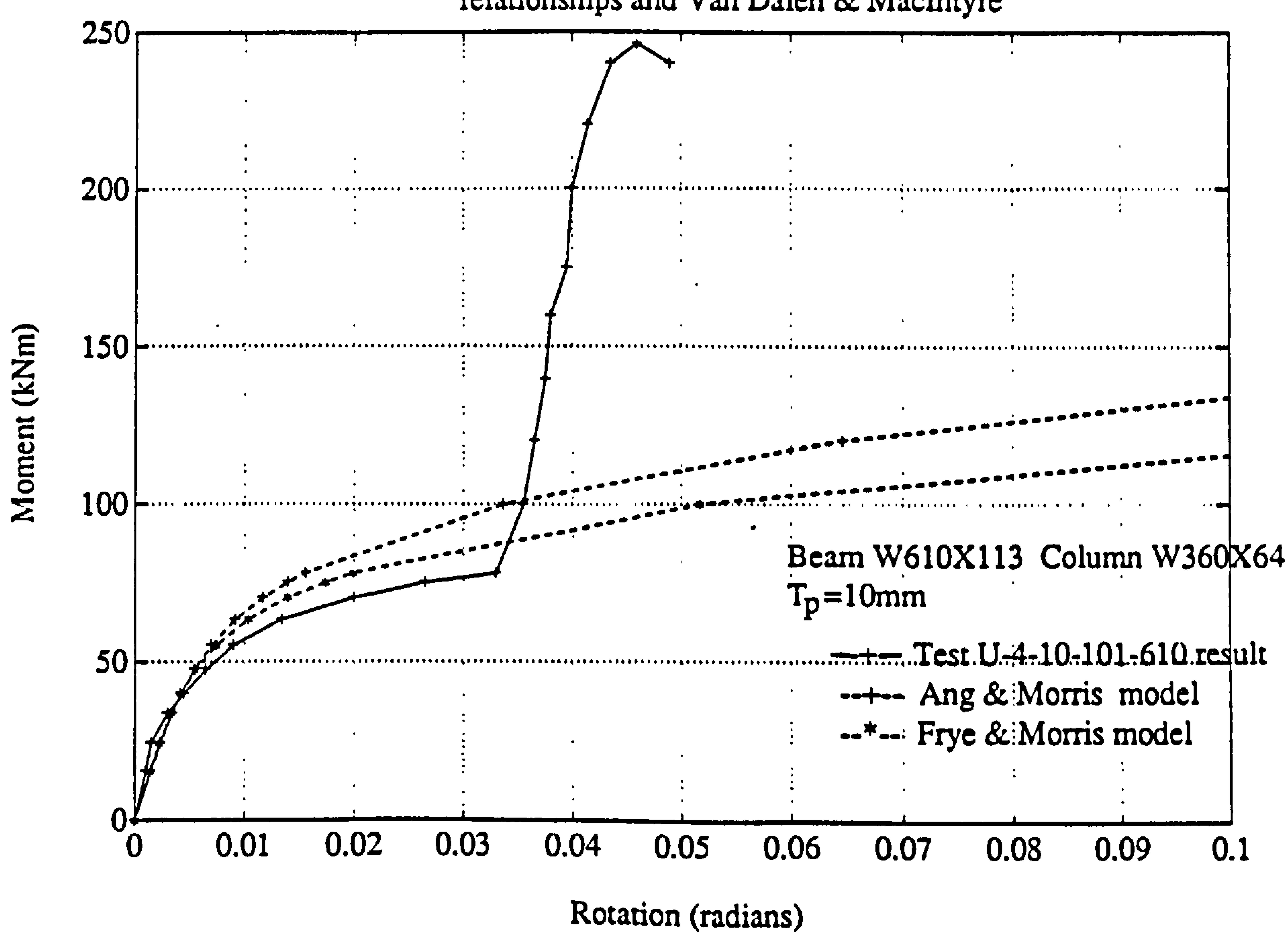


FIG.2-48 Comparison of moment-rotation curves for connections U-3-10-101-310 & Sommer 27

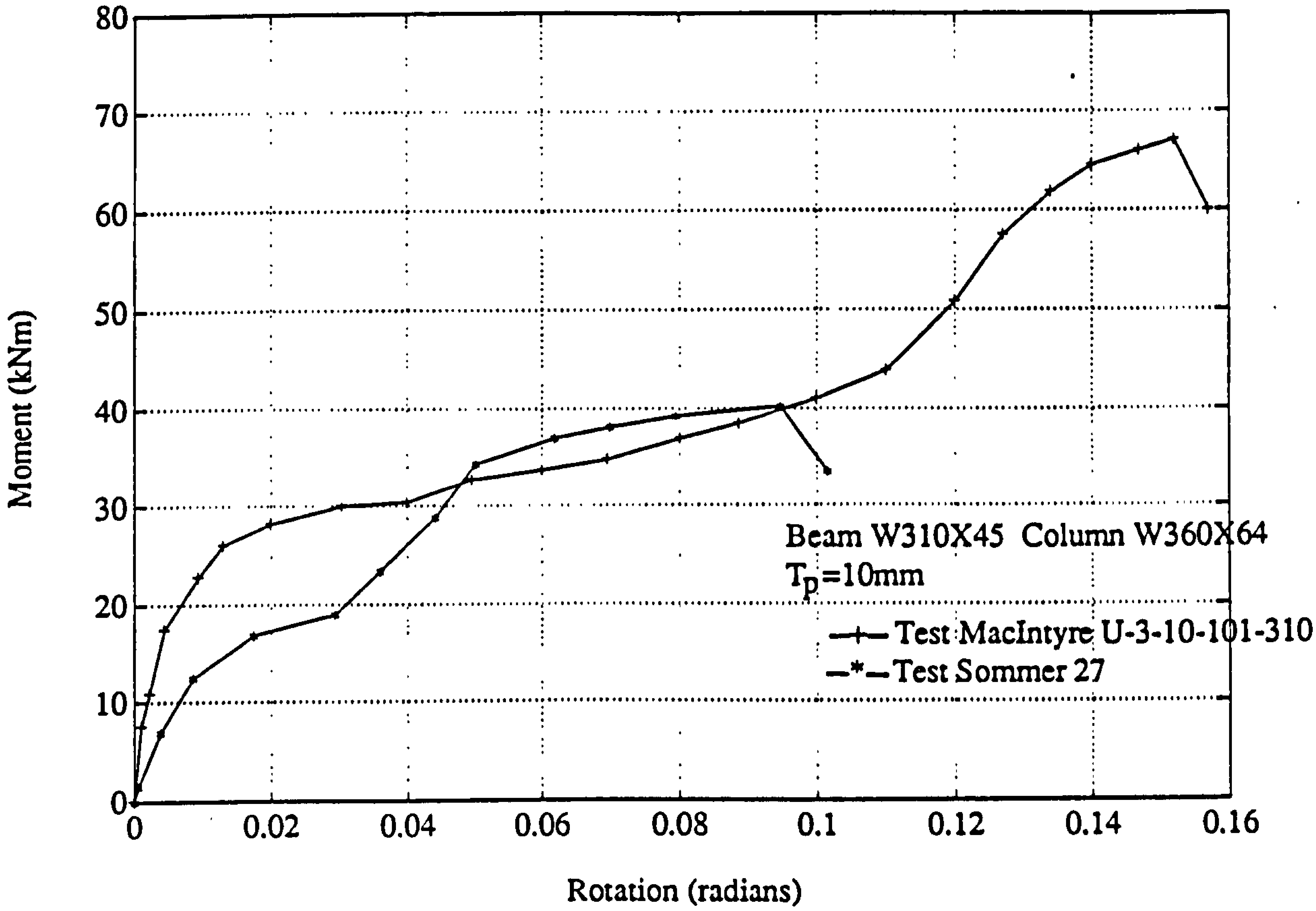


FIG.2-49 Comparison of moment-rotation curves for connections U-4-10-101-4600 & Sommer 11

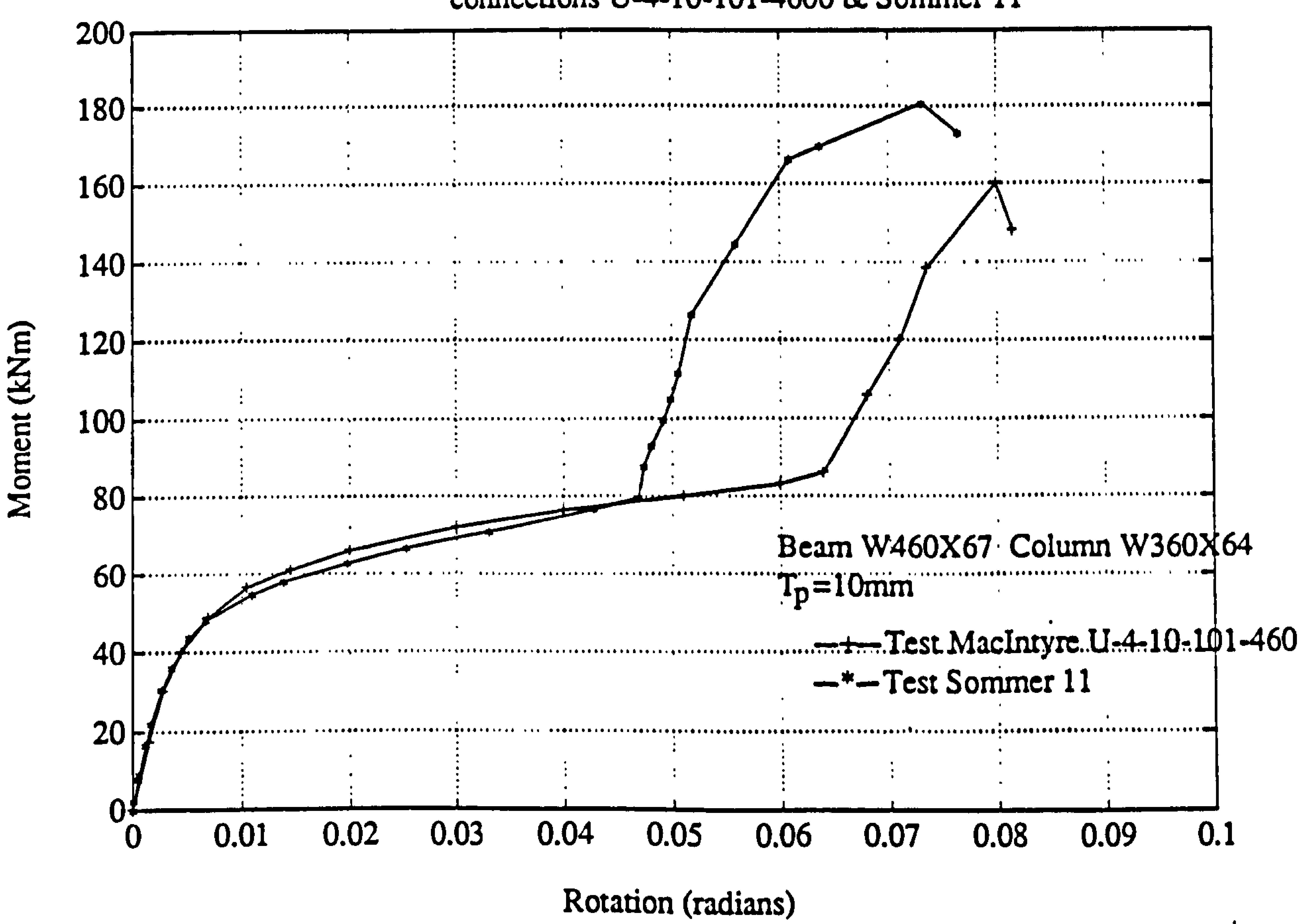
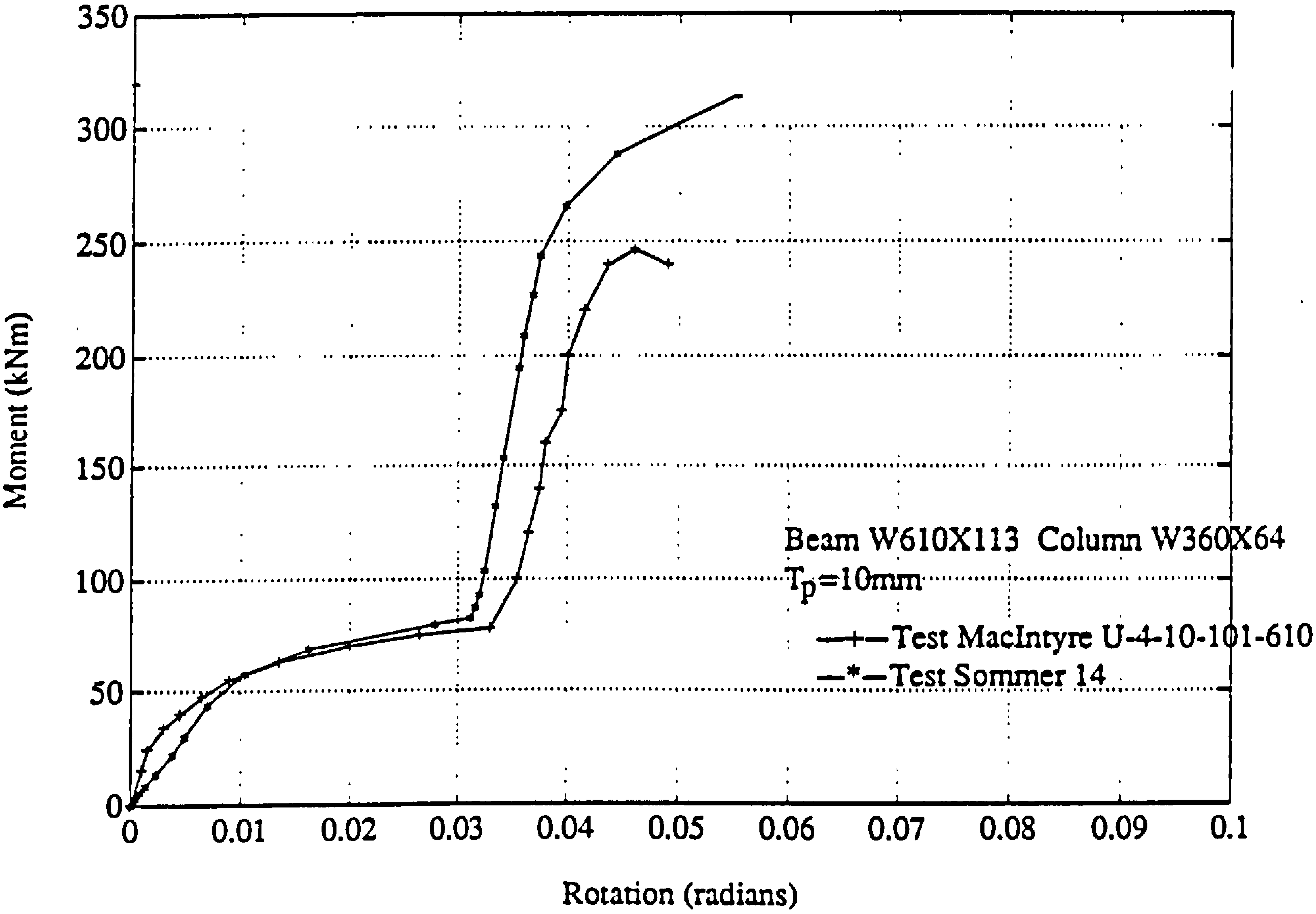


FIG.2-50 Comparison of moment-rotation curves for connections U-4-10-101-610 & Sommer 14



CHAPTER III

FLUSH END-PLATE CONNECTIONS

3.1 Introduction

As described in chapter one, one type of connection which has become increasingly common in steel construction with the advent of improved welding techniques is the end-plate connection. If the connection is to be a simple shear connection the plate is welded over the web of the beam only as shown in chapter two, whereas for a moment resisting connection the plate is welded over the entire depth of the beam, including the flanges. The plate is positioned perpendicular to the longitudinal axis of the beam with an equal number of holes on either side of the web.

A typical end-plate moment connection is composed of a predrilled or punched flat steel plate which is welded to the beam ends along the flanges and web in the fabricator's shop and the beam is erected and bolted in place to the column on site. In such connections, the loadings are normally transferred to the bolts through the end-plate. The behaviour of end-plate connections is complex and will depend upon the interaction of end-plate bending, bolts in tension, column flange bending and shear and direct force in the column web. Experimental evidence has shown that this form of connection is capable of transmitting significant moments. End-plate moment connections are classified as either flush or extended and may be with or without stiffeners. Stiffeners between the column flanges act to prevent the flexural deformation of the flange, which will influence the behaviour of the end-plate and the fasteners. Typical flush end-plate connection is shown in Figure 3.1. The extended end-plate connections will be the subject of the next chapter.

A flush end-plate connection is approximately equal in size to the beam depth, with all the bolts placed between the beam's flanges, as shown in Figure 3.1. Such connections are very popular in industry because of ease of fabrication and esthetic considerations, especially in roof details. Flush end-plate connections are typically used in frames subject to light lateral loadings and may be also preferred when a concrete slab is used in composite construction. Although, earlier experimental research carried out by many investigators [3.1, 3.2] on both extended and flush end-plate connections concluded that the behaviour and strength of these two connections differ greatly, the flush end-plate has received much less attention.

The notations for major parameters of flush end-plate connections, as shown in Figure 3.1, are identified as follows:

L_p = Plate depth,

B_p = Plate width,

T_p = Plate thickness,

T_{wb} = Beam web thickness,

D_b = Beam depth,

T_{fc} = Column flange thickness,

T_s = Column web stiffener thickness,

G = Distance between two lines of fasteners, referred as gauge distance,

P = Distance between two rows of fasteners, referred as pitch distance,

C_t = Distance from the tension side edge of plate to the outer surface of the tension side of the beam,

E_{1t} = Distance from the outer surface of the tension-side flange to the first row of tension-side fasteners,

E_{2t} = Distance from the outer surface of the tension-side flange to the second row of tension-side fasteners (if there is),

L_i = Distance from the centre of tension-side fastener holes to the centre of the compression side fastener holes,

C_c = Distance from the compression side edge of plate to the outer surface of the compression side of the beam,

E_{1c} = Distance from the outer surface of the compression side flange to the first row of compression side fasteners, and

E_{2c} = Distance from the outer surface of the compression side flange to the second row of compression side fasteners (if there is).

3.2 General behaviour and design recommendations

Due to the complexity of the behaviour of individual elements of the end-plate moment connection, it is difficult to predict the joint performance. The relationship between the moment acting on the connection and the rotation of the connection is best described by its moment-rotation curve. The moment-rotation curve for flush end-plate connections has virtually a curvilinear shape except for the most rigid connections, and for all connections at very small moments where the initial phase tends to be nearly linear. If the end-plate of a header plate connection is extended up to the beam flanges and welded to them (which gives rise to a flush end-plate connection), the performance of the connection improves significantly and the moment capacity may also be greater than 50% of the beam plastic moment. This is clearly shown in Figure 3.2a for the two tests carried out by Zandonini and Zanon [3.3] for a header plate (HPl-2) and flush end-plate (FPl-2) connections which were otherwise identical, even for the bolts arrangement. For the header plate, the ratio of the end-plate length to the beam depth in test (HPl-2) is 67%. In test

(HP13) in Figure 3.2b the ratio is about 82%. In a flush end-plate connection this ratio is approximately equal to unity. However for a header plate connection, the moment capacity is greatly influenced by the ratio of end-plate length to the beam depth as shown in Chapter two. In general for such connections, the greater the ratio, the more rigid the connection.

Initially the moment-rotation curve is almost linear with a slope which appears to be principally related to the end-plate thickness. After the initial phase, as the load increases, connection stiffness reduces due to softening somewhere in the connection. This is attributed to the separation of the end-plate from the column flange at the edge of the end-plate nearest to the beam tension flange [3.4, 3.5]. This separation in due to bending of the end-plate and the column flange. Experimental investigations [3.4, 3.6] reported that the first evidence of yielding usually appeared in the compression region of an unstiffened column web, on the end-plate at the tension bolts or on the column flange at the tension bolts. Once the connection has softened its stiffness becomes virtually linear again with a slope of about 1/40th of the initial value as described in references [3.7, 3.8, 3.9].

For stiffened column flange connections, a significant elongation appears in the bolts at the beam tension flange. Soon after first observation of yielding, other yielding often developed in other areas of localised tensile or compressive stresses. The location of these subsegment yield patterns appears to be influenced by the relative strength characteristics of the connection elements, the bolts pretension forces, and alignment of the specimen [3.4]. Before failure of the connection occurs, large flexural deformations of the end-plate and/or column flange take place.

As mentioned earlier, between the two phases, when the connection stiffness reduces due to softening of the elements of the joints, a rounded "knee" is produced in the

M-Φ curve. This knee occurred between 30 and 70% of the failure load of the connection and not at about 60% as claimed in references [3.7, 3.8]. The position in the M-Φ curve of the rounded knee is greatly influenced by the joint parameters such as end-plate thickness, column stiffening, column flange thickness for an unstiffened connection, pretension of bolts etc. Experimental evidence [3.4] indicates that the main concentration of tensile stresses was in the column web and to some extent in the beam web in the region of the tension bolts. To avoid concentration of tensile deformation in the column, column stiffeners should be used.

The predominant modes of failures experienced from test results on isolated flush end-plate connections are the following:

i) Failure of the bolts, which is defined either as the fracture of a bolt separating it into two parts or bolt stripping. Bolt fracture usually occurs for a thick end-plate where there is a negligible bending deformation of the plate (provided the column is strong enough to resist the external moment). In this case joint failure is sudden with little warning and the main rotation is due to bolt stretching.

ii) Failure of the weld, which is defined as the appearance of material cracks, near a weld. The cracks usually occurred in thin end-plate material adjacent to a fillet weld in the region of a tensile bolt. For connections with thin end-plates, the moment capacity of the connection will be generally less than that based on fracture of the bolts and much higher rotations can be achieved due to end-plate bending and column flange deformation.

iii) Failure of the column web, which is defined as the development of a column web buckle causing a decrease in the moment capacity of the joint. This mode of failure occurs usually only for connections with unstiffened column web.

iv) Failure of the beam compression flange, which is defined as the appearance of a buckle in this flange. Such a mode of failure reduces the moment capacity of the connection.

For a thin end-plate, prying forces are more evident and makes the structural behaviour of the joint more complicated.

It is important to understand the effect of each connection's parameters, such as thickness of end-plate, bolt arrangement, material properties etc., on the overall connection behaviour. Based on the collected experimental data, comparative moment-rotation curves are used to describe and illustrate the influence of the main parameters on the overall connection performance.

3.3 Parametric analysis:

A parametric analysis is developed from the comparative M-Φ curves to understand the contribution of each parameter on the joint behaviour.

3.3.1 Experimental data

The experimental data available from tests conducted on flush end-plate connections are tabulated in Table 3.1, with further details being provided in Appendix B, retaining the same ientification for each test as used by the original investigators. The numerical moment-rotation data is also presented, along with other pertinent identification. Unfortunately not all investigations have provided the material test results, preloading of the bolts and numerical M-Φ data despite private requests by the author in many cases. It is also stated in Appendix B whether the given numerical M-Φ data is provided from its original source or just a measure derived from the available plotted M-Φ curves. Data for this type of connection are available from 13 separate sources, covering a wide range of beam and column sizes, plate thicknesses, bolt arrangements and numbers, bolt sizes and grades and pretensioning forces. In total 100 tests have been identified on flush end-plate connections.

Ostrander [3.4] carried extensive experimental work on flush end-plate connections to study mainly the effect of end-plate thickness, column stiffeners and column section on the overall performance of the end-plate connection. The investigation of each of these effects was undertaken by varying separately each of the parameters, where possible. A total of 24 tests were conducted with 2 bolts at the tension side.

Zoetemeijer [3.11] conducted a test programme on bolted beam-to-beam connections to check that his proposed design rules would lead to connections that would satisfy the limit state of deformation as given in the Dutch regulation for constructional steelwork for serviceability and ultimate limit states. Flush end plate connections were used in only two tests. However, four additional tests were carried out on flush end-plate connections within a frame loaded to failure with increasing uniformly distributed load.

Zoetemeijer and Kolstein [3.12] tested two series of six flush end-plate connections at Delft University of Technology in 1975 to evaluate connection behaviour with different forms of column reinforcement. The test programme included two sizes of beams: an IPE400 and an IPE300 in each series of tests; 25 mm end-plate thickness was chosen. Column reinforcement by means of flange backing plates and/or compression web stiffeners, end-plate extended by 50 mm on the beam compression side to an unstiffened column flange with and without backing plate were investigated and compared to an unstiffened flush end-plate connection in each series of tests.

Zoctemeljer [3.13] later conducted an extensive series of tests on flush end-plate connections. A total of 23 tests were carried out with two types of sections: HE300A amd IPE400. The bolt arrangements and the plate thickness were varied, as well as the magnitude of the fillet weld which connected the end-plate to the beam. In five tests a second bolt row was placed below the bolt row adjacent to the flange and in four tests the bolt row adjacent to the flange comprised four bolts instead of the usual two. In all tests, the column flange was very stiff with respect to the flush end-plate.

Five tests on flush end-plate connections having two bolt rows in the tension region were carried out by Phillips and Packer [3.14]. These tests were identical except for end-plate thickness. The range of end-plate thickness was from 9.5 mm to 25.4 mm. The objective of the tests were mainly to determine the influence of end-plate thickness on moment-rotation characteristics and on the end-plate collapse mechanism. In addition, they studied end-plate connections with two rows of tension bolts, in order to examine the influence of the second row on the stiffness of the connection. They also suggested two failure mechanisms for end-plates with two rows of bolts, by which they determine the required thickness of the end-plate. On the basis of this test programme and on theoretical study, the authors concluded that:

i) Flush end-plates with two rows of two bolts in the tension region are suitable for semi-rigid construction.

ii) The second row of tension bolts is effective but to a much lesser extent than previously determined.

iii) The French design specifications best describe the connection's moment capacity where the bolt proof load can be reached before plate failure.

Bose [3.15] carried out an experimental investigation of three types of connections. Only one test specimen consisted of a flush end-plate, in which a 457 x 191 x 67

Universal Beam was welded to an end-plate 400 x 200 x 25 mm which in turn was bolted to a 305 x 305 x 97 Universal Column.

Morris and Newsome [3.6] undertook a series of four tests on bolted corner flush end-plate connections to compare the Morris stiffener with the more conventional form of stiffening and to investigate the shear-deflection behaviour in the column web and/or the column flange distortion produced by the bolt loads. An end-plate of 25 mm thick was chosen for the four tests in order that it would not deform significantly during test, and therefore the rotation of the end-plate could represent approximately the average shear-deformation of the panel.

Six tests on flush beam-column connections were carried out by Mann and Morris [3.5, 3.16]. The main purpose of this experimental investigation was to compare any changes in behaviour which arise from lack of fit of contact between an end-plate and a column flange produced by local curvature of the end-plate. The specimens were fabricated in two distinct groups, one with a thin end-plate (12 mm) and the other with a thick end-plate (20 mm), in order to investigate the effect of end-plate thickness in conjunction with lack of fit. In all tests, three rows of tension bolts, HSFG M22, were used. Within each group one specimen was fabricated to produce a near-perfect fit between the end-plate and column flange, the second specimen was made with an initial lack of fit of approximately 3 mm at the centre of the end-plate edge and the third specimen was tested with shims interposed between the end-plate and column flange. Lack of fit was found to be favourable because the contact forces between the connected parts were immediately underneath the load and any applied tensile force was then balanced by a decrease in the contact forces.

An extensive experimental programme was carried out by Tong [3.17] and Prescott [3.10], in order to investigate the overall behaviour of end-plate moment connection and to study the influence of the end-plate thickness, preload bolt force, spacing and positioning of bolt holes on the strength and stiffness of the connection. In total 12 isolated flush end-plate connections were tested. Beam sections, column sections, bolt diameters and bolt grades were chosen according to the reply of a survey of the structural steelwork industry [3.18, 3.19], with the aim to produce standardised connection details. The main purpose of standardising connections was to reduce the enormous number of parameters involved within a connection, i.e. Universal Beam and Column sections, end-plate dimensions and thickness, material properties, weld size and length, bolt size, grade, position and preload, etc.

Zandonini and Zanon [3.3] conducted a series of tests on end-plate connections, three of which were of the flush end-plate type. Three configurations were considered (with two, three or four bolt rows) for each type of connection, while the plate thickness was kept constant and equal to 12 mm. Bolts of grade 4.8, 20 mm in diameter, were used in the first two arrangements, while bolts of grade 6.8, 16 mm in diameter, were used in the connections with four bolt rows.

A series of tests on a variety of beam to column connections, ranging from very flexible web cleats to the relatively rigid extended end-plate, suitable for rectangular frames using I-section members, were tested by Davison [3.20]. The principal objective was the provision of moment-rotation data so that a comparative assessment of the performance of the different types, in terms of connection stiffness and moment capacity, could be examined. In total 25 connections were tested, eight of which were incorporated with various forms of lack of fit. Only on four tests where flush end-plate connections were used, three of which were connected to the column web and one to the column flange.

Two identical unstiffened flush end-plate connections were tested by Chakrabati [3.21] at the Building Research Establishment. The test specimen comprised two beams and a column to form a cruciform arrangement. Both connections used were a 290 x 154 x 12 mm end-plate welded to 254 x 146 x 37 UB and bolted by 6 x M16 grade 8.8 to 152 x 152 x 37 UC. Only the average M-Φ data recorded from the left and the right side connection of the column is tabulated in Appendix B.

As previously discussed in Chapter One, the moment-rotation relationship of a beam-to-column connection is one of the fundamental factors which influence the behaviour of the whole structure. To provide a better understanding of the behaviour of the connection and to identify the effect of varying significant parameters, the investigation of each of these problems, i.e. end-plate thickness, bolt's preload etc...., has been undertaken parametrically (where possible) by this author, comparing the M-Φ curves.

From the above collected data, no attempt has been made to investigate the effects of weld size or weld procedure on the moment-rotation characteristics and therefore this parameter is omitted from the next section.

3.3.2 Analysis of the moment-rotation characteristics

3.3.2.1 *Effect of end-plate thickness*

The moment-rotation curves for tests 3, 16, 17 and 5 of Tong [3.17] shown in Figure 3.3a, corresponding respectively to an end-plate thickness of 12, 15, 20 and 25 mm, clearly indicate that increase in end-plate thickness leads to an increase in connection stiffness. Similar results are shown in Figure 3.3b from Phillips and Packer [3.14] test results. The moment capacity of a connection also increased with end-plate thickness provided that bolt fracture or weld failure did not occur at ultimate moment. The effect of increasing the ultimate moment capacity of the connection by increasing the end-plate thickness based on experimental test results is discussed in references [3.5, 3.10, 3.13, 3.14, 3.16, 3.17 and 3.22]. Although the end-plate thicknesses are 20 and 25 mm for tests 17 and 5 respectively of Figure 3.3a, the ultimate moments in both tests are approximately the same (about 165 kNm) because of the mode of failure of the connection which was due to the fracture of the first tension bolt row (the bolt row immediatelty adjacent to the beam tension flange), for both tests.

As earlier discussed in section 3.2, failure modes and rotations change as end-plate thickness changes. Thin end-plates tend to fail by a development of cracks occurring in the end-plate material adjacent to a fillet weld in the region of the tension bolts, whereas thicker end-plates result in beam or column failure for unstiffened connections and bolt fracture in other cases. The tests of references [3.10, 3.17] permit some conclusions to be drawn on the effect of end-plate thickness on connection behaviour. These are similar to those previously mentioned, i.e. for thin end-plates the majority of rotation comes from the bending of the end-plate, while for thick end-plates the rotation is due to bolt yielding.

3.3.2.2 *Effect of beam depth*

Unlike the header plate, for the flush end-plate connections, the depth of the connection is directly related to the beam depth because in such connections the end-plate is approximately equal in size to the beam depth. Therefore, a deeper beam section requires a deeper end-plate connection which attracts higher moment. This is clearly shown in Figure 3.4 for a IPE300 and IPE400 beam sections from Zoetemeijer and Kolstein's test results [3.12].

3.3.2.3 *Effect of bolt number and arrangement*

Figure 3.5a compares two tests 3 and 12, identical except for the horizontal bolt cross-centres distance (gauge distance) shows that a decrease in gauge distance leads to an increase in connection stiffness and in the ultimate strength of the connection. Many investigators [3.5, 3.13, 3.16, 3.17] recommend that bolts be placed as close as physically possible to the beam flange and web and within the recommended distances (i.e. edge distance of bolts, and gauge distance) by the code of practice and the standard steel section book to reduce prying forces [3.5].

Zoetemeijer [3.11, 3.13, 3.24, 3.25], Zoetemeijer and Munter [3.23] and Zoetemeijer and Kolstein [3.12] have studied both theoretically and experimentally the behaviour of flush end-plate connections, with particular emphasis on bolt number and arrangement in references [3.12, 3.13]. The main conclusions to be drawn from these studies are:

i) *Effect of second bolt row adjacent to the web*:

Figures 3.6a, 3.6b, 3.6c and 3.6d show a family of moment-rotation curves where the only variable is the presence of a second bolt row below the bolt adjacent to the web. In Figure 3.6c and 3.6d the effect of the second bolt row is not very significant in the M-Φ curve. However in Figures 3.6a and 3.6b, the presence of a second bolt row has a significant effect on the M-Φ curve and on the moment-capacity of the connection. Based on test results [3.12, 3.13], Zoetemeijer [3.13] suggested that the additional bolt should be placed within the yield line mechanism of the bolt adjacent to the flange in order to contribute significantly to the overall M-Φ behaviour and consequently to the strength of the connection.

In test specimens [3.13], a second bolt below the bolt adjacent to the web had almost no effect on moment-rotation curves for the 300 mm deep beams. However, for the 400 mm deep beams the contribution of the additional bolt row below the bolt adjacent to the web has a significant effect on the connection stiffness and on the ultimate moment.

ii) *Effect of a second bolt adjacent to the flange*:

Figures 3.7a and 3.7b show that adding a second bolt adjacent to the flange (i.e. the bolt row adjacent to the flange comprised four bolts instead of the usual two) had almost no effect on the connection stiffness.

As a result of the above tests, it is concluded that the joint flexibility increases with an increase in either horizontal spacing of the bolts (i.e. gauge distance) and/or the distance of the top row of bolts below the beam flange. The presence of second bolt row below the bolt adjacent to the flange also increases the connection stiffness. Therefore, the above mentioned parameter should be accounted for when determining the moment-rotation characteristics to achieve an accurate prediction.

3.3.2.4 *Effect of bolt preload*

In 1966, Hellquist [3.26] studied the effect of bolt pretension on the moment-rotation characteristics of semi-rigid end-plate connections of moderate end restraint capacity, and the behaviour, under load, of high strength bolts within the connection. He tested three connections, each assembled from two 10WF21 beams, and 8WF28 column stub with half inch (12.7 mm) thick end-plate and three quarter inch (19.05 mm) diameter A325 high strength bolts. The average pretension force applied to the bolts was different in each specimen: 13.9 kips per bolt (62 kN), 7.8 kips per bolt (35 kN) and finger tight. Although, in all his tested specimens the average bolt pretension was below the minimum required pretension force of 28 kips (about 124.5 kN) as specified by CSA standard S16-1969, he concluded that the behaviour of an end-plate connection is greatly affected by the amount of bolt pretension. "The greater the pretension, the more rigid the connection" [3.26].

Comparison of moment-rotation curves between two distinct groups tested by Prescott [3.10], one with all the bolts hand tightened corresponding to a preload force of 25 kN and the other with a preload of 100 kN, shown in Figures 3.8a and 3.8b for two end plate thicknesses, clearly indicates that the amount of bolt pretension significantly affects the M-Φ characteristics of end-plate connections. Figure 3.8 and other experimental results [3.10, 3.17, 3.26] revealed that the stiffness of the end-plate connection in the range up to working load is greatly influenced by bolt pretension but the failure load of an end-plate connection appears to be independent of the magnitude of the bolt pretensioning force. The latter statement is due to the fact that at the ultimate loading stage of bolted joints, the connected faces separate and therefore the behaviour of the preloaded connection would be similar to the behaviour of the identical connection without preload. Thus, they will have similar ultimate moment and rotation [3.17].

The effect of bolt preload for thin end-plates, such as the case for 12 mm end-plate thickness, test 19 of Prescott [3.10] not shown here, is insignificant. This is because, for the thin end-plates, the main connection rotation comes from the end-plate and there is no contribution from the extension of the bolts in the preloaded connection, until the bolt preload has been overcome. Based on experimental results, bolt preload has more effect on the thicker end-plate connections. Therefore, when testing end-plate connections to determine their M-Φ relationships, it is important to measure the bolt pretensioning force because it is an important parameter and should be included in the prediction equation of moment-rotation relationship.

3.3.2.5 *Effect of column section*

Curves showing the influence of column section for flush end-plate connections are shown in Figure 3.9. In general, an increase in column section weight results in an overall increase in the rigidity of the connection and in failure moment and a decrease in the joint rotation at failure.

Based on experimental results [3.4], for column section where the column flange thickness is greater than the end-plate thickness, there is a small increase in connection stiffness as shown in Figure 3.9a. However, for end-plate thickness greater or approximately equal to the column flange thickness, there is a considerable increase in connection stiffness by increasing the column flange thickness as shown in Figure 3.9b. Test 16 in Figure 3.9b fails at a low moment due to the fact the end-plate and column flange are too thick and therefore the end-plate and/or the column flange are unable to deform plastically. Fracture of a bolt was the mode of failure of this specimen.

3.3.2.6. *Effect of column stiffeners*

If the flush end-plate is considered as pinned connection, which is sometimes the case in industry, and it is designed by assuming to carry only or mainly shear, then the use of column stiffeners is irrelevant. However, experimental evidence [3.4, 3.10, 3.17] has shown that flush end-plate connection must be considered as moment-connection and can transmit a moment as high as 85% of the plastic moment of resistance of the beam. Thus, the column stiffeners could be used to increase the connection rigidity by:

i) protecting column web against deformation by using compression column stiffeners and

ii) preventing flexural deformation of the column flanges by using tension column stiffeners.

In several tests, column with very heavy flanges [3.13] or with a large amount of web stiffening designed to minimise column flange deformation [3.5, 3.14, 3.16] were used. Backing plates were also used in tests of reference [3.12], giving the same effect as web stiffeners.

Figure 3.10 shows families of moment-rotation curves where the only variable in each test is the thickness of the column stiffener. Two stiffener thicknesses are tested, 1/4 inch (6.35 mm) and 3/8 inch (9.525 mm) and compared with an instiffened connection. The main conclusions to be drawn from these above mentioned M-Φ curves and other experimental results carried out by Ostrander [3.4] are the following:

i) Apparently for a thick column flange and thin end-plate, the stiffeners do not increase the column rigidity and consequently there is an insignificant increase in connection stiffness (see Figure 3.10a). Whether using 1/4 inch (6.25 mm) or 3/8 inch (9.525 mm) thick column stiffeners, there is negligible changes in shape of the M-Φ curve. In such connections, the use of column web stiffeners has no effect on change of the mode of failure as confirmed by the tests carried out by Ostrander [3.4].

ii) However for connection with end-plates thicker than the column flange, such as Figure 3.10b, the use of column stiffners increases substantially the connection stiffness. In addition to the increase of connection stiffness, with reference to Figure 3.10b, column stiffeners affect the failure mode and reduce the rotation of the connection at failure. The column stiffeners reduce the deformation of the column web and confine deformation of the column flange, therefore resulting in a much stiffer connection than for an unstiffened arrangement.

Figure 3.11 examines the effect of different types of column stiffening on moment-rotation characteristics. Three different ways of stiffening the column are compared with an unstiffened connection. In the first set of curves (Figure 3.11a) the beam section is an IPE 300 and in the second set of curves (Figure 3.11b) the beam section is an IPE 400, with an end-plate thickness of 25 mm for both sets. Connections with backing plate on compression side against the column flanges tends to increase the moment capacity of the connection and to reduce the connection rotation at failure. Initial connection stiffness appears to be not influenced by the use of backing plate. Connections with end-plates extended on the compression side tend to have higher moment-capacity than connections with compression web stiffeners. Increasing end-plate depth beyond the depth of the beam produces little if any change on the M-Φ curve for an IPE400 beam section (see Figure 3.11b). However for an IPE300 beam section, the effect of extending the end-plate on compression side is more pronounced as it is shown in Figure 3.11a for test 17 from reference [3.12].

3.3.2.7. *Effect of yield stress*

One of the most important parameters in elastic-plastic analysis is the static yield stress denoted here by F_y. It may be influenced by the position from which the specimens are taken. For I-sections, BS 4360 permits these to be cut from either the web or the flange. For the usual structural steels according to BS 4360, a set of values of design strength is given in Table 6 of BS 5950. These values differentiate between different grades, thicknesses and types of section. For example, values of yield stress in Table 6 of BS 5950 range from 245 to 275 N/mm² for grade 43 steel. However, in all specimens tested, the material yield stress determined by the tensile tests are in excess of the specified mimimum yield stresses of the material. Figure 3.12 from reference [3.10] shows the importance of incorporating realistic material properties into the M-Φ prediction equation. The ratio of the actual yield stress to the nominal one in this figure is about 20%.

Figure 3.12 also shows the effect of end-plate yield stress on M-Φ characteristics. Similarly, actual yield stress values for the column section are also important especially when the column web buckling or flange deformation are significant factors. As discussed in section 3.2, whithin a joint after the initial phase which is almost linear elastic behaviour, connection stiffness reduces due to yielding of one or more of the constituent elements of the connection. Thus the softening of the connection is not due only to the yielding of the end-plate. Therefore, the accuracy of the M-Φ prediction equation beyond the elastic range can be much improved if the actual yield stresses obtained by tensile tests are used for the theoretical modelling, rather than by using the nominal values.

As a result of this parametric analysis with reference to Figure 3.1 it is important to incorporate the following main connection parameters in the prediction equation of M-Φ relationship:

i) end-plate thickness, T_p

ii) horizontal gauge distance of the bolts, G

iii) beam depth, D_b

iv) distance from the tension top beam flange to the centre of the first row of bolts, E_{1t}

v) distance from the tension top beam flange to the centre of the second row of bolts, E_{2t}

vi) proof load of the bolts, P_f

vii) preload of the bolts, P_l

viii) column flange thickness, T_{fc}, for unstiffened column web

ix) actual yield stress of the end-plate, F_{yp}

x) whether the connection is connected to a column flange, a column web or a girder web, and

xi) whether the connection is connected to a stiffened or unstiffened column.

Despite the fact that the actual yield stress parameters for column web, column flange and beam flange, F_y, have not been discussed in this section because of lack of test data for evidence, the author felt necessary to include these parameters in predicting M-Φ curves.

3.4 Analytical representation of moment-rotation curves

3.4.1 Frye and Morris's model

Frye and Morris [3.27] proposed a formula for prediction of the moment-rotation behaviour of end-plate connections. They recognised the difference in connection behaviour when the column is stiffened versus an unstiffened column and provided different equations for each case. Frye and Morris's prediction equation makes no distinction between flush and extended end-plate connections. The equation is developed on the same principles as those for header plate connections by Sommer [3.28] previously discussed in Chapter two, section 2.4.1. The curve-fit experimental M-Φ data was based on a data base that consisted mainly of Ostrander's [3.4] test results.

For end-plate connections without column web stiffeners, the rotation is given by the following polynominal function:

$$\Phi = 1.83 \times 10^{-3}(kM) - 1.04 \times 10^{-4} (kM)^{3} + 6.38 \times 10^{-6} (kM)^{5} \quad (3.1)$$

where the standardisation constant, k, is given by:

$$k = L_b^{-2.4} \;.\; T_p^{-0.4} \;.\; T_{fc}^{-1.5} \quad (3.2)$$

For end-plate connections with column web stiffeners, the rotations is given by:

$$\Phi = 1.79 \times 10^{-3} (kM) + 1.76 \times 10^{-4} (kM)^{3} + 2.04 \times 10^{-4} (kM)^{5} \quad (3.3)$$

where the standardisation constant, k, is given by:

$$k = L_b^{-2.4} \quad T_p^{-0.6} \quad (3.4)$$

The connection parameters in the above equations L_b, T_p and T_{fc} are respectively distance between extreme bolts centre, end-plate thickness and column flange thickness as shown in Figure 3.1. The parameters L_b, T_p and T_{fc} are in inches and the moment, M, is in kips.in and not in a dimensionless form as claimed by Frye and Morris [2.28]. The rotation, Φ, is in radians.

In metric units, the prediction equation for end-plate moment connection without column web stiffeners is given by:

$$\Phi = 8.92 \times 10^{-3} (kM) - 12.03 \times 10^{-3} (kM)^{3} + 17.53 \times 10^{-3} (kM)^{5} \quad (3.5)$$

and

$$k = L_b^{-2.4} \cdot T_p^{-0.4} \cdot T_{fc}^{-1.5} \quad (3.6)$$

For end-plate moment connection with column web stiffeners, the M-Φ relationship is given by:

$$\Phi = 2.60 \times 10^{-3} (kM) + 5.37 \times 10^{-4} (kM)^{3} + 1.31 \times 10^{-3} (kM)^{5} \quad (3.7)$$

and

$$k = L_b^{-2.4} \; T_p^{-0.6} \quad (3.8)$$

where in equations (3.5) to (3.8);

Φ is rotation of the connection in radians,

M is moment of the connection in kNcm, and

k is standardisation constant in $cm^{-4.3}$ in equ. (3.6) and in $cm^{-3.0}$ in equ. (3.8).

The exponent of the column flange thickness, T_{fc}, given in equ. (3.2) and (3.6), differs from that in reference [3.28] where the exponent was given as 1.1 to improve the predicted connection rotation [3.29, 3.30]. Goverdhan [3.29] proposed to take L_b, vertical distance between extreme bolt centre, as the beam depth and not as the distance between the extreme bolt centres to get a better approximation of the true behaviour of the connection. According to Frye and Morris [3.28], the maximum deviation of the calculated connection rotation from the experimentally measured value is 3% for the unstiffened connections and 6% for flush end-plate connection with column stiffeners.

3.4.2 Johnson and Law's model

Johnson and Law [3.31] have presented an analytical method for M-Φ relationship of a beam-column joint for a composite frame, in which continuous slab reinforcement and bolted end-plate connections are used. The behaviour of the joint is idealised as bilinear M-Φ curve and is based on the assumption that the steel connection is taken as a component.

The coefficient for the initial stiffness of a flush end-plate connection is derived by assuming that the interaction between different joint components affects only negligibly the response of each single component considered in isolation, and therefore the behaviour of the whole joint may be obtained simply by superimposing the stiffness of the components constituting the joints (i.e. bolts, column flange and end-plate). This is shown in Figure 3.13 and it is expressed as follows:

$$C = \frac{1}{0.5\ [(1/C_b) + (1/C_c)]\ \lambda + [1/C_e]} \qquad (3.9)$$

where C_b, C_c and C_e are respectively the stiffnesses of the bolts, the column flange and the end-plate.

The factor λ arises from the eccentricity of the connecting element with respect to the top flange of the beam and is equal to $(D/h)^2$, with the following assumptions:

$$C = \frac{T}{\Delta}, \quad M = TD \text{ and } \theta = \frac{\Delta}{D}$$

The elastic stiffness of the steel joint is equal to CD^2 i.e.

$$K_{joint} = CD^2 \tag{3.10}$$

With the assumption that prying forces and preload force of the bolts are neglected, the stiffness of the bolt is given by:

$$C_b = E\,A_b/g \tag{3.11}$$

where E is Young's modulus for steel, A_b is the total bolts area in the tensile region and g is the grip length.

The stiffness C_c is determined by assuming the column flange as an infinitely long cantilever plate subjected to a transverse concentrated load (see Figure 3.14) and it is given by:

$$C_c = \pi\,\bar{D}/Ka^2 \tag{3.12}$$

where

$$\bar{D} = E\,t_{fc}^3/12(1-v^2) \tag{3.13}$$

where

a is the outstanding portion of the column flange (see Figure 3.14),

c is distance between point load and clamped edge (see Figure 3.14),

t_{fc} is column flange thickness,

v is poisson ratio of the material and

k is dimensionless coefficient obtained from the exact solution by Jaramillo [3.32].

The stiffness C_e is determined by assuming the end-plate fully restrained by the bolts and to deform as a simple cantilever of length (D-h) (see Figure 3.13d) with a point load T at its end, hence its stiffness is given by:

$$C_e = 3EI/(D-h)^3 \tag{3.14}$$

where I is the second moment of area of the end-plate at the level of the top line of bolts.

The plastic moment of the connection is based on a yield line analysis developed by Packer and Morris [3.33]. The yield load of the flange, T_y, is given by:

$$T_y = t_{fc}^2 \times F_y \; \{\pi^2 + (4n - D')/2m\} \tag{3.15}$$

The notation and the assumed yield line mechanism are shown in Figure 3.14 and consequently the moment capacity of the joint is given by:

$$M_p = T_y \; . \; h \tag{3.16}$$

In the above equations, the original notation is used throughout the text to avoid any possible confusion.

3.4.3 Kukreti, Murray and Abolmaali's model

More recently, Murray et al [3.34] presented a method based on finite element modelling, to develop analytically the moment-rotation relationship for a bolted steel end-plate connections. Experimental testing of selected specimens was conducted to confirm and modify accordingly the finite element model and associated computer analysis procedure. A parametric study was conducted and the data collected were regressed to develop a prediction equation characterising the general behaviour of the connection [3.34].

The 2-D dimensional finite element mesh configuration used in the analytical model is shown in Figure 3.15 where the bolt centreline nodes are constrained against vertical displacement. The bolt area in any row (top or bottom) is assumed to be uniformly spread across part of the width as shown in Figure 3.15. The width of equivalent rectangular bolt area, g_b, is given by:

$$g_b = 1/3\ (F_{by}/F_{yb})/(A_B/d_b) \qquad (3.17)$$

where

F_{by} is yield stress of the beam material,

F_{yb} is yield stress of the bolt material,

A_B is beam-cross sectional area, and

d_b is nominal bolt diameter.

The effective stress-strain relationship for the elements of the end-plate, beam flanges and web is taken to be elastic-perfectly plastic to model the nonlinear material behaviour. The bolt shank material is assumed to follow a bilinear stress-strain curve. The maximum distortional energy theory (Von Mises) is used to check yielding of an element. In order to conduct a parametric study, a total of 256 different beam cross sections were chosen to limit the geometric variables to practical ranges.

The primary objective of the parametric study is to develop a prediction equation for the maximum end-plate separation, δ, at the level of the beam tension flange for the computation of the connection rotation ($\theta = \delta/h$ where h is the beam depth) and it was chosen as the quantity to correlate the 3-D and 2-D results.

A 2-D finite element analysis was performed on 50 of the 256 selected beam sections. The obtained results for maximum end-plate separation, defined as

$\pi_a = \delta$, were statistically analysed using a multiple regression analysis computer program and the prediction equation obtained was:

$$\pi_a = e^{-8.336}(t_p/h)^{7.620} \cdot (p_f/h)^{-6.932} \cdot (t_w/h)^{-0.501}$$

$$(t_f/h)^{-0.032} \cdot (b_p/h)^{2.885} \cdot (d_p\ /h)^{-0.849} \qquad (3.18)$$

$$(g_b/h)^{-0.519} \cdot (P_f^3/b_p\ t_p^3)^{3.053} \cdot M^{1.356}$$

where:

t_p is the end plate thickness,

p_f is the pitch of the bolt (i.e. distance from top of the flange to the centreline of the bolt),

t_w is the beam web thickness,

t_f is the beam flange thickness,

b_p is the end-plate width,

d_p is the nominal bolt diameter,

h is the beam depth and

g_b is given by equation (3.17).

Based on the comparison of the experimental results, with a 3-D model and the 2-D model of the maximum end-plate separation, it was decided to use the 2-D model with a correction factor of 1.5.

Therefore, the end-plate separation is given by:

$$\delta = 1.5\ \pi_a \qquad (3.19)$$

The notation, θ, of the flush end-plate connection at any load level is given by:

$$\theta = \frac{\delta}{h} \qquad (3.20)$$

Substituting equation (3.18) and (3.19) into equation (3.20), the rotation is given by:

$$\theta = CM^{\beta} \qquad (3.21)$$

where

$$C = (359 \times 10^{-6}) \,.\, (p_f)^{2.227} \,.\, (h)^{-2.672} \,.\, (t_w)^{-0.501} \,.\, (t_f)^{-0.032}$$

$$.\, (d_p)^{-0.849} \,.\, (g_b)^{-0.519} \,.\, (b_p)^{-0.168} \,.\, (t_p)^{-1.539} \qquad (3.22)$$

The value of β for a single bolt row in tension flush end-plate connection is 1.356 given by the regression analysis results. The units used in equ. (3.21) and (3.22) for length is inches and for moment is kip.ft. The values of the exponents for beam depth, h, for beam flange thickness, t_f, and for end-plate width b_p, differ from that in reference [3.34] where the exponent were respectively incorrectly given as -2.616, -0.038 and -0.218. The maximum allowable moment for a connection is given by equations (3.11) and (3.12), adopting a value of δ = 0.01 in for maximum allowable end-plate separation at the level of the beam tension flange. The difference between the maximum experimental applied moment and the predicted moment by equ. (3.18) and (3.19), checked in reference [3.34] against the eight test results on flush end-plate connections welded to two beams and tested by Srouji [3.35] as splice connections under pure moment, lies within the range of -5% to 20% (the negative sign indicates lesser predicted value). In the above equations, original notation is used throughout the text.

3.4.4 Proposed model for M-Φ prediction equation

As earlier described in Chapter One, different models have been introduced, namely a mathematical model, a mechanical model and a finite element model. Despite the fact the latter is the most suitable technique for conducting such complex investigations, it is considered as inadequate at present because of time and cost involved, as well as uncertainty over assumptions inherent in the analysis. The mechanical model is quite difficult to set up because of the complexity involved in the connection/joint behaviour and the large number of physical parameters such as geometrical and mechanical properties. However, the most commonly used approach to describe the moment-rotation relationships is to curve-fit the experimental data with simple mathematical expressions. such model can represent extremely well any shape of M-Φ curve and it is recognised that this form of approach appears to have been most successful and most widely applied. Comparison between the reviewed methods and test results will be discussed in length in section 3.5. As a result of this comparison, there is no suitable equation for predicting the moment-rotation relationship for the flush end-plate connection. Hence an analytical model is developed based on the use of the collected M-Φ data and on the curve fitting technique.

The moment-rotation of a flush end-plate connection represents a nonlinear relationship between the moment applied to connection, M, and to connection rotation, Φ, as defined in Chapter One Section 1.4. By considering the properties of this moment-rotation curve, it is evident that a nonlinear mathematical expression for the curve must satisfy the following requirements:

1. $M = 0$ at $\Phi = 0$ i.e. the curve passes through the origin

2. $dM/d\Phi = k_i$ at $\Phi = 0$ i.e. the slope of the curve at the origin is equal to the initial elastic stiffness of the connection "k_i".

3. $dM/d\Phi = 0$ as $\Phi \to \infty$ i.e. as the rotation becomes large, the curve assymptotes to M_p and levels off.

4. The mathematical expression should be capable of recognising the typical shape of an identical fairly straight portion, followed by a region of gradually reducing slope, leading to a shallow final part.

5. The mathematical expression should be of relatively simple form.

6. At any load before failure of the connection is reached, the connection stiffness, which is represented by the slope of the M-Φ curve, should be positive. A negative slope is physically unacceptable and it may cause numerical difficulties in the analysis of framed structures using the tangent stiffness formulation which will be described in Chapter 5.

A prediction equation for the moment-rotation relationship for the flush end-plate connection has been developed by the present author based on the available experimental test data using a regression analysis. A polynomial model represents the M-Φ behaviour reasonably well but the main drawback is that the nature of a polynomial is to peak and trough within a certain range causing the connection stiffness to become negative at some values of M and therefore the requirement number 6 cited above is not satisfied.

The best fit for individual tests is to assume the connection rotation given by the following expression:

$$\Phi = C_1 M^{C_2} \exp(C_3 M) \tag{3.23}$$

where C_1, C_2 and C_3 are functions of the connection parameters. However equation (3.23) does not lead to succesful results because a regression is carried out on each factor C_i separately, causing the error to build up from the three C_i factors. Another simple alternative model of M-Φ relationship which gives satisfactory results was developed. This has the following form:

$$M = C_1 \frac{\Phi}{C_2 + \Phi} \qquad (3.24a)$$

or

$$\Phi = C_2 \frac{M}{C_1 - M} \qquad (3.24b)$$

As the prediction equation (3.24) does not predict successfully the connection rotation over the complete loading curve, it has been decided to split up the M-Φ curve into two parts. The first part is up to the knee which is taken at about 60% of the moment capacity of the connection [3.7] and the second part from the knee up to the end of the loading curve. However this has been found to give an odd curve around the knee, i.e. discontinuity at the knee, because as described in section 3.2 of this chapter, the knee does not occur precisely at 60% of the moment capacity of the connection. As it was shown in Chapter Two (section 2.2) that the maximum rotational deformation for a beam-to-column connection in a frame designed to meet the recommended deflection limits would be 0.0226 radians for steel grade 43, it is decided to fit equation (3.24) only up to a rotation of 0.023 radians. This model, given by equation (3.24), which is particularly well suited for characterising the moment-rotation relationship for flush end-plate connection, also represents a nonlinear relationship between M and Φ that levels off, or saturates, as Φ increases.

Linear and nonlinear regression analysis are available to fit these equations (such as equations (3.23) and (3.24)) to experimental data directly such as in references [3.36, 3.37]. However, a simple alternative is to use mathematical manipulation to transform the equation into a linear form. Then simple linear regression can be employed to fit the equations to data. Equation (3.24a) is linearised by inverting it to yield:

$$\frac{1}{M} = \frac{C_2}{C_1} \cdot \frac{1}{\Phi} + \frac{1}{C_1} \qquad (3.25)$$

Thus, a plot of $1/M$ versus $1/\Phi$ will be linear, with a slope of C_2/C_1 and an intercept of $1/C_1$.

The linearised version model is shown in Figure 3.16. In its transformed state, the model is fitted using linear regression analysis in order to evaluate the empirical values, C_1 and C_2, based on experimental measurements of M for various values of Φ.

Firstly, it is required to calculate C_1 and C_2 from the empirical data. This is accomplished by using equ. (3.25) in its linear form for each separate collected test specimen. A regression analysis computer program GO2CAF [3.38] is used to perform this simple linear regression with dependent variable $1/M$ and independent variable $1/\Phi$. Having calculated the empirical values C_1 and C_2 for each test specimen, then a regression analysis is carried to fit these constant values by a multi-variable power function. The power equation to be evaluated is of the form:

$$C_i = p_1^{\alpha_{i,1}} \cdot p_2^{\alpha_{i,2}} \cdot p_3^{\alpha_{i,3}} \cdots\cdots p_k^{\alpha_{i,k}} \qquad (3.26)$$

where $\alpha_{i,k}$ is the value of the power corresponding to the model parametric C_i and to the connection parameter k. The parameters p_1, p_2 ... p_k are identified by the parametric analysis (section 3.2).

A multiple linear regression of Ln (i.e. logarithms) - transformed data provides a means to evaluate such relationship (i.e. equ. (3.26)). Taking the Ln of this equation (3.26) yields:

$$\mathrm{Ln}\, C_i = \alpha_{i,1}\, \mathrm{Ln}\, p_1 + \alpha_{i,2}\, \mathrm{Ln}\, p_2 + \alpha_{i,3}\, \mathrm{Ln}\, p_3 + \ldots + \alpha_{i,k}\, \mathrm{Ln}\, p_k \quad (3.27)$$

In this form, equation (3.27) is suited for multiple linear regression because Ln C_i is a linear function of Ln p_k. The $\alpha_{i,k}$ parameters, regression coefficients, are estimated from the data using the multiple regression analysis computer program GO2CJF [3.38].

Evaluation of model parameter

Because of considerable differences in behaviour, bolted end-plate connections must be considered in two groups:

1. Stiffened connections = these are connections that are provided with column web panel stiffeners (Figure 3.1) and can, with proper design, transmit the full moment capacity of the adjacent beam and column. Generally, failure of the column web is avoided in such type of connections and the connection/joint parameter T_{fc} (i.e. column flange thickness) can be dropped from equation (3.26).

2. Unstiffened connections = these connections are similar, but are not provided with column web panel stiffeners. Generally, they cannot transmit the full moment capacity of the adjacent beam or column. Failure of the weaker connection elements governs the moment capacity of these connections.

The model developed here is limited to flush end-plate connections to unstiffened columns with either a single row of two rows of bolts in the tension region and up to rotation of 0.023 radians. The M-Φ equation is assumed to be of the form:

$$\Phi = C_2 \frac{M}{C_1 - M} \tag{3.24b}$$

and the empirical factors C_i are given by

$$C_i = D_b^{\alpha_{i,1}} \cdot T_p^{\alpha_{i,2}} \cdot T_{fc}^{\alpha_{i,3}} \cdot G^{\alpha_{i,4}} \cdot E_{1t}^{\alpha_{i,5}} \cdot E_{2t}^{\alpha_{i,6}} \cdot F_{yp}^{\alpha_{i,7}} \cdot F_{yc}^{\alpha_{i,8}} \cdot F_{yb}^{\alpha_{i,9}} \cdot P_f^{\alpha_{i,10}} \cdot P_l^{\alpha_{i,11}} \tag{3.28}$$

where the parameters P_k are defined at the end of section (3.1) and the power's values, , are given below.

$\alpha_{1,1}$	= + 0.870	$\alpha_{2,1}$	= - 2.385
$\alpha_{1,2}$	= + 0.917	$\alpha_{2,2}$	= + 0.281
$\alpha_{1,3}$	= + 1.299	$\alpha_{2,3}$	= + 0.631
$\alpha_{1,4}$	= - 0.652	$\alpha_{2,4}$	= - 0.013
$\alpha_{1,5}$	= - 0.919	$\alpha_{2,5}$	= - 0.561
$\alpha_{1,6}$	= - 0.006	$\alpha_{2,6}$	= - 0.270
$\alpha_{1,7}$	= + 0.379	$\alpha_{2,7}$	= - 0.258
$\alpha_{1,8}$	= + 2.004	$\alpha_{2,8}$	= - 0.234
$\alpha_{1,9}$	= + 0.090	$\alpha_{2,9}$	= + 0.643
$\alpha_{1,10}$	= - 0.233	$\alpha_{2,10}$	= + 0.221
$\alpha_{1,11}$	= + 0.240	$\alpha_{2,11}$	= + 0.391

The regression analysis was performed on the collected numerical data expressed in units of kN and cm. Consequently to get the right use of the M-Φ prediction equation, equations (3.24) and (3.28), the same units should be employed. The units are therefore cm for lengths, kN/cm² for stresses, kN for loads, radians for rotations and kNcm for moments.

Based on data reported with a complete detail of geometric parameters and material properties, the test specimens were selected. The data used are given in Table 3.2. The constants C_1 and C_2 are empirical values based on experimental rotation Φ for various values of moment M for the data range given in Table 3.2. From Figure 3.16, the empirical constant C_1 can be regarded as a maximum attainable moment and not as the moment capacity of the connection. In other words, C_1 is an upper bound of the moment capacity of the connection. Once the applied moment exceeds the value C_1 the connection rotation given by equation (3.16) becomes negative and cannot be used to define the moment-rotation characteristics of the connection.

Despite the fact that 57 M-Φ curve data are collected for unstiffened flush end-plate connections, only 13 test results were used in developing equation (3.16). This is due to the enormous difficulties met in getting full data from the investigators. The private communication of the present author with several investigators revealed that for many specimens tested, the material yield stresses determined by the tensile tests and the measurement of the bolt preload forces were not carried out.

3.5 Comparison of experimental and analytical M-Φ curves

3.5.1 Flush end-plates without column web stiffener

3.5.1.1 *Frye and Morris's prediction equation*

Although experimental evidence [3.3, 3.10, 3.17, 3.20] shows that the behaviour of a flush end-plate differs greatly from an extended end-plate connection, Frye and Morris [3.27] proposed only one equation (3.1) and they make no distinction between the two types of connections. In equation (3.1) L_b is defined as the vertical distance between extreme bolt centres. Followed the suggested definition by Goverdhan [3.29] as the depth of the beam, it is apparent that taking L_b as it is defined by Frye and Morris leads to very flexible connection behaviour. However, if L_b is increased, the M-Φ curve is stiffened and the curve-fitting is improved. In all the M-Φ test curves checked against the predicted curves given by equ. (3.1) using L_b as the beam depth, the predicted M-Φ relationships were much more stiffer than the one using L_b as defined by Frye and Morris [3.27]. The plots with L_b as defined in reference [3.27] are not included in the thesis; the reader can refer to reference [3.29]. Due to the presence of a minus sign in the polynomial form of equ. (3.1), reverse curvature could happen in some cases which in turn apparently causes the connection to stiffen with increased moments. As the Frye and Morris [3.27] prediction equation is based on a data base that consisted on 11 tests of Ostrander [3.4] and one of Sherbourne [3.2], a complete check of the comparison of M-Φ curves for the 11 tests is shown in Figures 3.17 to 3.27. In general, for those tests which were included in the regression analysis, equ. (3.1) gives better results for end-plate thickness less than or equal to the thickness of the column flange as shown in Figures 3.19, 3.21, 3.22, 3.24 and 3.27 corresponding respectively to tests 4, 11, 12, 17 and 23 of Ostrander [3.4]. This is due to the fact for thin end-plate connections, much higher rotations can be achieved due to the ability of the end-plate to bend which in turn gives a flexible connection; the prediction equation represents better the behaviour of flexible flush end-plate connections. For these tests the predicted connection rotations exceed the experimental values by up to 50% and not by 3% as claimed by Frye and Morris [3.27]. Despite the fact that the other tests by Ostrander, where end-plates are thicker than the column flange, were used in deriving the model, the predicted moment-rotation relationships err greatly and cannot be used to represent the connection behaviour, except for test 13 shown in Figure 3.23 which failed due to bolt fracture.

Figures 3.28, 3.29 and 3.30 show further comparisons of moment-rotation characteristics of a flush end-plate connection to an unstiffened column versus the Frye and Morris method of prediction. These tests were not included in the data base of Frye and Morris used to generate equ. (3.1). For test specimens 12 and 18 of reference [3.12], the model predicts satisfactory the connection behaviour, as is shown in Figures 3.28 and 3.29. However for test JT/12 of reference [3.20] the predicted M-Φ curve (Figure 3.30) is unsatisfactory and inadequately represents the connection behaviour, even at very low moments. This is despite the fact the end-plate thickness is nearly double than the column flange thickness. Based on these comparisons and others not included in the thesis, the predicted curves cannot represent the moment-rotation relationships with any consistency.

3.5.1.2 *Johnson and Law's prediction equation*

Johnson and Law [3.31] proposed a method for the prediction of the initial-stiffness and plastic moment capacity of flush end-plate connections to unstiffened column webs. It is assumed that the interaction between different joint components only negligibly affects the response of each single component considered in isolation, and the behaviour of the whole joint may be obtained simply by superimposing the flexibilities of the components of the joint, (i.e. member's elements, connecting elements, fasteners). A bilinear model is developed based on the evaluation of the initial stiffness and the ultimate moment capacity. Such an approach, apparently rough, presents the advantage of drawing the attention only to two basic parameters and not all the moment-rotation curve range. In general, a bilinear model is acceptable only if a small joint rotations are likely to occur and providing the linear initial connection stiffness is reasonably well predicted.

Several plots are presented in Figures 3.17 - 3.30 to show how the various prediction equations compare with the experimental data. Examination of these plots indicates that the linear representation of connection stiffness as suggested by Johnson and Law [3.31] is apparently unsatisfactory and inadequate for end-plate thinner than the column flange and for connections to flanges with approximately the same thickness. However, for end-plates thicker than the column flange, the model predicts reasonably well the initial connection stiffness. To get a closer approximation of the initial connection stiffness and with the intention of predicting the initial connection behaviour on the flexible side rather than on the rigid side, Goverdhan [3.29] suggested that the initial connection stiffness should be represented by the greater of:

1. The initial stiffness as proposed by Johnson and Law [3.31]; and
2. (M_p/0.01 rad.) with M_p as defined by equ. (3.16).

After careful checking of several plots not included in this thesis with the proposed modified model, the author believes that the true initial behaviour could be better represented by the maximum of the first above requirement and (M_p/0.005) radians).

As the moment capacity of the connection is derived from the assumption that column flange yielding is the main criterion in predicting M_p, equ. (3.16) is in considerable error when the failure mode of the connection is other than column flange yielding. Therefore, in the checked tests the predicted moment capacity of the connection is usually under estimated in the range of 2% to 30% in comparison with the experimental values for connection when the end-plate is thicker than the column flange. However, for tests 9 and 13 of Ostrander [3.4] and test JT/12 of reference [3.20] the differences are outside this range. In test 9 the relative error is an under-estimate of 37%, explained by the fact that the connection failed by a beam compression buckle which reduces the actual moment capacity of the connection. However, in test 18 the failure mode of the connection is a column web buckle and the error in predicting M_p is an over-estimate of 4%. Although, the ratio of end-plate thickness over column flange thickness for test JT/12 of Davison [3.20] is 1.765 (i.e. very light column), the relative error in predicting the moment capacity of the connection is an under-estimate of 42%. Column 4 of table 3.3 illustrates the ratio of the predicted moment capacity of the connection to the maximum applied moment for the test specimens shown in Figure 3.17 to 3.30.

3.5.1.3 *Murray et al prediction equation*

The prediction equation (equ. (3.21)) provided by Murray et al [3.34] is based on a finite element method analysis exclusively for flush end-plate connections to unstiffened columns, with only one row of bolts in tension. The model predicts a much higher initial stiffness than what is provided by the connections studied. Certainly a power function such as equ. (3.21) does not relate the connection rotation and the moment by a linear relationship but the prediction equation does not follow at all the curvilinear shape behaviour of a flush end-plate connection. In all checked test results shown in Figures 3.17 to 3.27, the predicted connection behaviour is very stiff over the whole moment-rotation curve even at a very high load. The analytical model does not take into account column dimensions or properties. The rotation of the connection is surely influenced by the magnitude of the prestressing force of the bolts and by the horizontal distance between the bolts which are not included in this model. The ratio of the moment values predicted by Murray et al to the maximum applied moment (i.e. $M_{experimental}$) for the 11 test specimen of Ostrander [3.4] are indicated in column 5 of Table 3.3. The difference between predicted (i.e. equ. (3.18) and (3.19)) and experimental results lies within the range -30% to -70% (negative sign implying lesser predicted values).

3.5.1.4 *Proposed prediction equation*

The developed model based on regression analysis of the collected data is applicable for unstiffened flush end-plate connections with two or four-bolts at the tension flange. Experimental moment-rotation curves for flush end-plates without web stiffener have been compared against the proposed prediction equation in Figures 3.17 to 3.30. The developed model (equ. (3.24)) predicts extremely well the initial connection stiffness. The prediction equation follows the experimental curve with acceptable deviation on both the flexible and stiff side, generally up to rotation of a least 0.020 radians, except for test specimen 4 of Ostrander [3.4] shown in Figure 3.19 where the end-plate thickness is 0.25 in (6.35 mm). This is due to the fact that the parameter T_p for this particular test is at the end of the range of data used to derive the coefficients C_1 and C_2 of equ. (3.24). In Figure 3.29, the predicted M-Φ curve approximates extremely well the connection stiffness up to a moment of about 50% of the moment capacity of the connection; then it diverges from the experimental curve. There is no apparent reason or physical interpretation to explain this discrepancy of divergence at high loads. In all other cases, the maximum difference between predicted connection rotation and experimental results lies within the range -20% to + 30%.

The model does not provide a rigorous method for the prediction of the moment capacity of the connection but it gives an upper bound which limits the use of the moment-rotation relationship. Column 6 of Table 3.3 represents the ratio of predicted upper bound moment to the maximum applied moment. It can be seen that this value varies between -9% to +51% (negative sign implying a lesser predicted value). In all checked results, this value is over-estimated (i.e. an upper bound) except for tests 12 of Zoetemeijer and Kolstein [3.12] and test 18 of reference [3.4].

In view of the complexities involved in the problem, these comparisons show that the moment-rotation relationship, equ. (3.24), developed for the unstiffened flush end-plate connection with either a single row or two rows of bolts in the tension region, is considered to be satisfactory. The empirical value C_1 cannot be used to determine the moment capacity of the connection but it can be regarded as a limiting value of the applied moment for which equ. (3.24) can be applied. As the failure moment of an end-plate connection appears to be independent of the magnitude of the preload force of the bolts (see section 3.3.2.4), in predicting the moment-capacity of the connection, P_l should not be considered. However, in the prediction equation the empirical values C_1 and C_2 are determined to represent the behaviour of the flush end-plate connections over all the range of loading and therefore the parameter P_l (preload force of the bolts) should be included.

3.5.2 Flush end-plates with column web stiffeners

3.5.2.1 *Frye and Morris's prediction equation*

Only one equation is available for the prediction of the moment-rotation curve for flush end-plate connections with a column web stiffened by a horizontal plate. The derivation of this equation is mainly based on Ostrander's [3.4] tests on flush end-plate connections. The behaviour of the connection is represented only by two parameters, the end-plate thickness T_p and the vertical distance between the extreme bolt centres L_b. Despite the fact that L_b is increased by assuming it as the beam depth (following the proposal by Goverdham [3.29]) in order to improve the approximation of the connection behaviour, the predicted M-Φ curves by Frye and Morris [3.27] are all very flexible when checked against the experimental results.

The plots in Figures 3.31 to 3.34 compare this method of prediction against all of Ostrander's [3.4] experimental curves. Figures 3.31a and 3.31b represent a set of three tests on each figure where the variables on each test are thickness of the end-plate and/or thickness of the column web stiffener. The prediction method does not result in any difference in connection behaviour for tests 2 and 5 shown in Figure 3.31a and also for tests 7 and 8 shown in Figure 3.31b. This is because, the equation does not take into account of the thickness of the stiffeners which affect slightly the shape of the M-Φ curve. The predicted curves for tests 7 and 8 both correspond to 0.25 in (6.35 mm) end-plate thickness and represent extremely well the connection behaviour.

Figure 3.33 consists of a set of moment-rotation curves for W12 x 27 beam section and W8 x 24 column section. The parameter varied is the end-plate thickness. The Frye and Morris prediction equation, equ. (3.3), approximates well the experimental M-Φ curves.

Figure 3.32 consists of a set of moment-rotation curves for three end-plate connections to W8 x 40 column. The only parameter varied is the end-plate thickness. For Ostrander's test 24 shown in Figure 3.34 the column section is W8 x 48. Despite the fact that tests 14, 15, 16 and 24 were used in the least square method in deriving the Frye and Morris prediction equation, the predicted curves shown in Figures 3.32 and 3.34 with a relatively heavy column section comparing to W8 x 24, errs greatly on the flexible side as the end-plate thickness increases from 0.375 in (9.525 mm) to 0.625 in (15.875 mm).

As mentioned earlier in section (3.3.2.4) the moment-rotation curve for flush end-plate is dependent on the preload force of the bolts. Figure 3.35 and 3.36 show a set of three end-plate thicknesses on each figure where the difference between figures is the preload force. The predicted curves for the two tests with the same end-plate thickness are exactly the same because Frye and Morris does not account for preload force of the bolts.

As a result of this comparison, the Frye and Morris curve errs substantially on the flexible side and showed no reverse curvature. The equation approximates the test curve extremely well for end-plate thickness not exceeding 10 mm and with shallow beam like W10's.

3.6 Conclusions

A data bank has been created, from which conclusions have been drawn. This data bank shows that commonly used configurations of flush end-plate connections exhibit a semi-rigid behaviour and should be analysed as semi-rigid connections. As such connections are very popular in modern construction due to ease of fabrication and erection and do not exhibit neither a rigid nor a pinned behaviour, a need is felt to develop an equation to predict the moment-rotation relationship. The behaviour of flush end-plate connection with or without column web stiffeners is affected by many parameters. The main connection parameters are identified. The available experimental isolated connection tests confirmed that the moment-rotation relationship of such connections is highly nonlinear and cannot be represented at all accurately by a linear or bilinear model. Therefore, it would be reasonable to consider the moment-rotation relationship to be represented by a non linear curve.

Column stiffeners increase the rigidity of flush end-plate connections in two ways: Firstly, the stiffeners adjacent to the compression flanges of the beam(s) restrained the deformation of the column web and secondly, the stiffeners adjacent to the tension flanges of the beam(s) confined and restrained the deformation of the column flanges. Therefore it is essential to consider the difference in behaviour of stiffened and unstiffened connections in developing the moment-rotation relationships.

Flush end-plate connections to unstiffened columns

The Frye and Morris prediction equation makes no distinction between flush end-plate and extended end-plate connections. Certainly, taking L_b as the depth of the connected beam, as suggested by Goverdhan, gives better approximations of the moment-rotation relationships but the effect of bolt pitch is lost. Since the equation does not take account of the type, size, number, arrangement and preload force of the bolts nor material properties of the connected elements, the equation should not be used for predicting the moment-rotation relationships.

Since the prediction equation of Johnson and Law is based on the evaluation of the initial stiffness and the ultimate moment capacity of the connection, this bilinear model represents a rough approach to estimate the behaviour of the connection over the entire M-Φ curve range. The prediction of the initial connection stiffness is unsatisfactory and inadequate for end-plates thinner than the column flanges and for connection to flanges with approximately the same thickness. For failure modes of the connection other than yielding of the column flange, the predicted value of the moment capacity is in considerable error. It is recommended not to use the bilinear model to represent the curvilinear behaviour of the flush end-plate connections.

The proposed prediction equation by the author is reasonably accurate for connections with end-plate thickness greater than 6mm and for beam depths ranging from 250 mm to 400mm. The experimental curves show a good agreement with the analytical curves up to a minimum connection rotation of 0.02 radians. The initial connection stiffness is reasonably well estimated. The equation predicts a higher moment capacity than that provided by the connection.

Flush end-plate connections to stiffened column

The Frye and Morris prediction equation for end-plate connections with column stiffeners does not differentiate between flush end-plate and extended end-plate connections. From the comparisons presented in the preceding section, the predicted curves are not able to approximate the experimental curves with any consistency. The Frye and Morris prediction equation for these connections is inaccurate and unacceptable even at very low moments. This is due in part to the fact the equation does not take account of bolt arrangements (i.e. gauge distance, pitch distance and number of bolts) and the bolt pretension force, especially in the range of working loads and rotations. The rotation of the connection is surely influenced by the type of fastener used to connect beam-to-column. The effect of material properties of the connected elements are neglected which may not be justifiable. Certainly the maximum deviation of standardised curve from experimental curve is not 6% as claimed by the investigators and cannot be used to represent the moment-rotation relationship for such connections.

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Table 3.1 Available experimental data for flush end-plate connections

Reference of experimental data	Date	Country of origin	Number of tests	Type of fastener	Test type	Comments on M-Φ curves
Ostrander [3.4]	1970	U.S.A.	13 unstiffened + 11 stiffened	Cruciform load in column	$\frac{3}{4}$ in diam A325 bolts	Provided for each test
Zoetemeijer [3.11]	1974	Netherlands	2 unstiffened	Cantilever	M20 & M22 G10.9 bolts	Provided for each test
Zoetemeijer [3.11]	1974	Netherlands	4 unstiffened	Portal frame	M22 Gr Gr 10.9 bolts	Provided for each test
Zoetemeijer & Kolstein [3.12]	1975	Netherlands	8 unstiffened + 4 stiffened	Cruciform load in column	M24 Gr 8.8 bolts	Provided for each test
Zoetemeijer [3.13]	1981	Netherlands	23 unstiffened	Cantilever	M24 Gr 8.8 Bolts	Provided for 6 tests + 7 supplied privately
Phillips & Packer [3.14]	1981	Canada	5 stiffened	Cantilever	$\frac{7}{8}$ in diam A325 bolts	Provided for each test
Bose [3.15]	1981	U.K.	1 unstiffened	Cruciform load in column	M20 Gr 8.8 bolts	Provided for each test
Morris & Newsome [3.6]	1981	U.K.	4 stiffened	Cruciform	$\frac{7}{8}$ in diam bolts	Provided for each test
Man & Morris [3.5, 3.16]	1981	U.K.	6 stiffened	Cruciform load in column	M22 HSFG bolts	Only moment-deflection curves provided

Table 3.1 continued

Tong [3.7]	1985	U.K.	6 stiffened	Cruciform load in column	M20 Gr 8.8 bolts	Provided for each test
Zandonini & Zanon [3.3]	1986	Italy	3 unstiffened	Cantilever	M20 Gr 4.8 + M16 Gr 6.8 bolts	Provided for each test
Davison [3.20]	1987	U.K.	1 unstiffened	Cruciform load in bolts	M16 Gr 4.6	Provided for each test
Prescot [3.10]	1987	U.K.	6 stiffened	Cruciform load in column	M20 Gr 8.8	Provided for each test
Chakrabati [3.21]	1987	U.K.	2 unstiffened	Cruciform load in column	M16 Gr 8.8 Bolts	Provided for each test

Table 3.2 Data range for various geometric parameters and material properties

Test identification \ Parameter	D_b (cm)	T_p (cm)	T_{fc} (cm)	G (cm)	E_{1t} (cm)	E_{2t} (cm)	F_{yp} (kN/cm²)	F_{yc} (kN/cm²)	F_{yb} (kN/cm²)	P_f (kN)	P_l (kN)
Test 1 [3.4]	25.146	1.270	1.176	8.890	6.350	0	31.30	35.10	28.48	126.3	185.7
Test 3 [3.4]	25.146	0.952	1.176	8.890	6.350	0	31.37	35.10	28.48	126.3	176.6
Test 4 [3.4]	25.146	0.635	1.176	8.890	6.350	0	32.06	35.10	28.48	126.3	182.6
Test 9 [3.4]	25.146	1.905	1.176	8.890	6.350	0	22.06	35.16	29.51	126.3	187.5
Test 11 [3.4]	30.378	0.952	1.417	10.160	6.350	0	43.37	33.51	39.99	126.3	184.8
Test 12 [3.4]	30.378	1.270	1.417	10.160	6.350	0	29.23	33.51	39.99	126.3	166.3
Test 13 [3.4]	30.378	1.587	1.417	10.160	6.350	0	28.20	33.51	39.99	126.3	185.7
Test 17 [3.4]	30.378	0.952	1.011	10.160	6.350	0	43.37	35.50	39.99	126.4	185.5
Test 18 [3.4]	30.378	1.270	1.011	10.160	6.350	0	29.23	35.50	39.99	126.3	179.9
Test 19 [3.4]	30.378	1.587	1.011	10.160	6.350	0	28.20	35.50	39.99	126.3	184.3
Test 23 [3.4]	30.378	1.587	1.738	10.160	6.350	0	28.20	30.06	39.99	126.3	185.7
Test JT/12 [3.20]	25.500	1.200	0.681	7.600	5.500	10.50	29.04	26.86	30.82	34.8	45.0
Test 12 [3.12]	40.000	2.500	1.200	10.000	5.000	12.00	29.40	30.00	30.65	201.0	79.0
Test 18 [3.12]	30.000	2.500	1.200	10.000	4.500	11.50	29.40	30.00	31.10	201.0	120.0

Table 3.3 Comparison of maximum applied moment of test results with predicted values

Test designation	Maximum applied moment (KNm)	Failure Mode M_{exp}	(1) $\frac{M_{predicted}}{M_{exp}}$	(2) $\frac{M_{predicted}}{M_{exp}}$	(3) $\frac{M_{predicted}}{M_{exp}}$
Test 1 [3.4]	61.9	BM	0.69	0.37	1.27
Test 3 [3.4]	54.9	*	0.77	0.30	1.09
Test 4 [3.4]	29.8	*	1.43	0.35	1.40
Test 9 [3.4]	67.8	BM,C	0.63	0.63	1.49
Test 11 [3.4]	72.2	W	1.21	0.33	1.23
Test 12 [3.4]	91.2	B	0.96	0.42	1.06
Test 13 [3.4]	100.1	B	0.87	0.50	1.20
Test 17 [3.4]	65.3	W	0.53	0.36	0.98
Test 18 [3.4]	74.8	C	0.47	0.51	0.95
Test 19 [3.4]	70.8	C,B	0.49	0.70	1.23
Test 23 [3.4]	100.9	B	0.86	0.49	1.25
Test 12 [3.12]	164.4	*	0.73	-	0.91
Test 18 [3.12]	84.4	*	1.04	-	1.51
Test JT/12 [3.20]	24.7	*	0.58	-	1.03

Index to failure modes:
B = Bolt failure
C = Column web buckle
* = Failure not reached
BM = Beam compression flange buckle
W = End plate weld failure

Index:
(1) $M_{predicted}$ = moment predicted by Johnson and Law
(2) $M_{predicted}$ = moment predicted by Murray et al
(3) $M_{predicted}$ = moment predicted by author

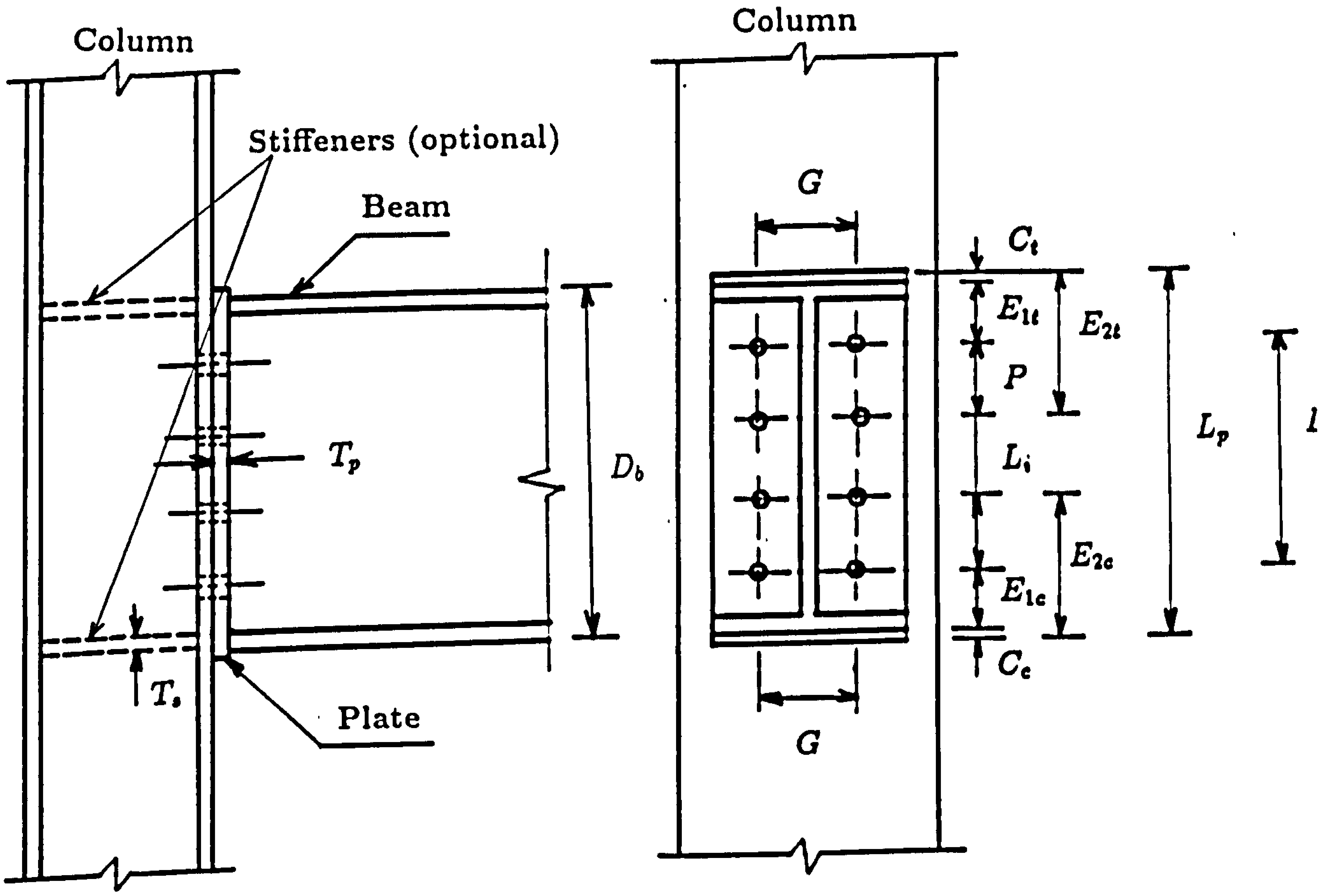
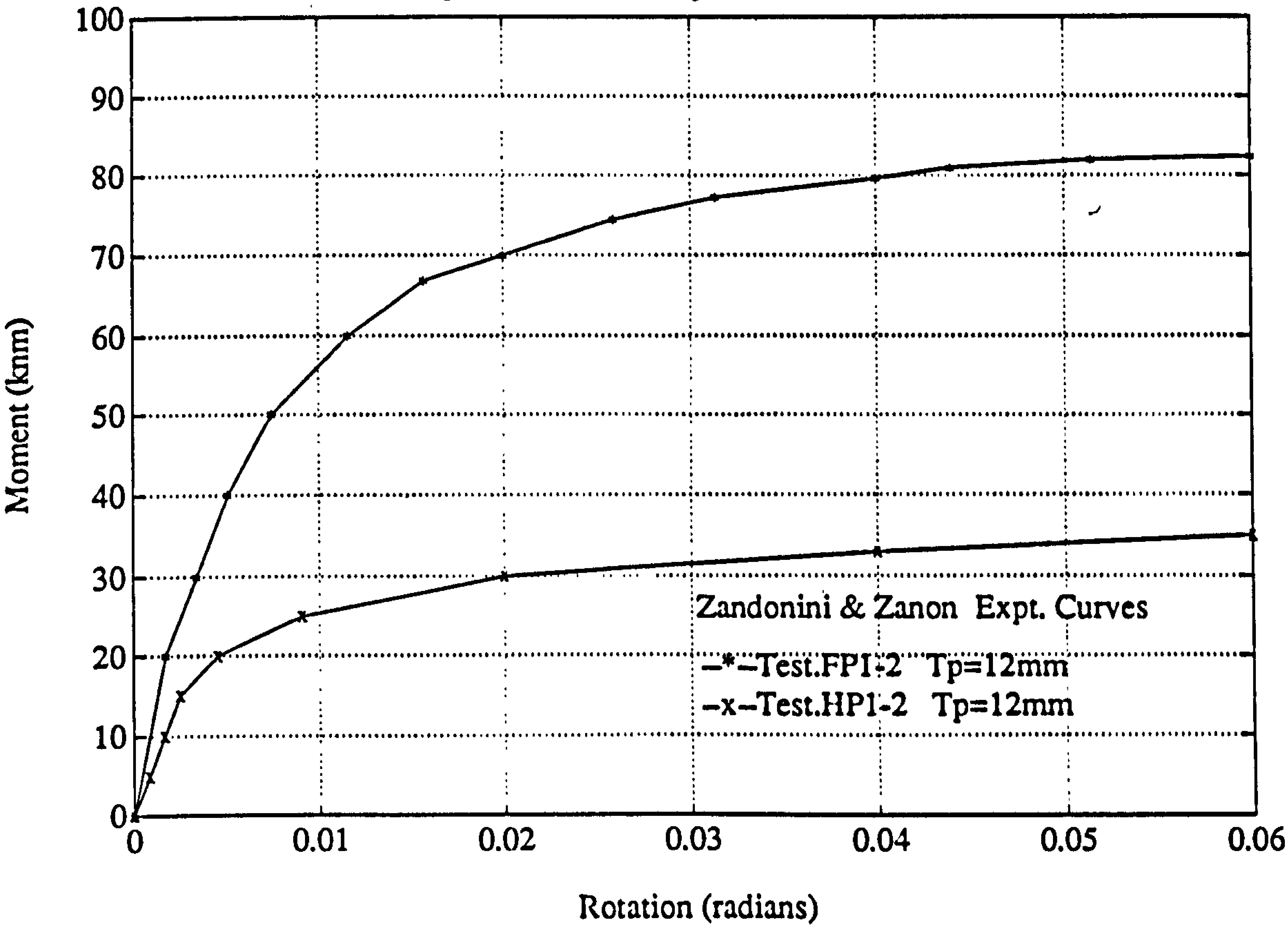


Figure 3.1 Typical flush end-plate connection.

Fig.3-2 Comparison of M-Φ curves for a fush-end plate and a header plate connection.

(a)



(b)

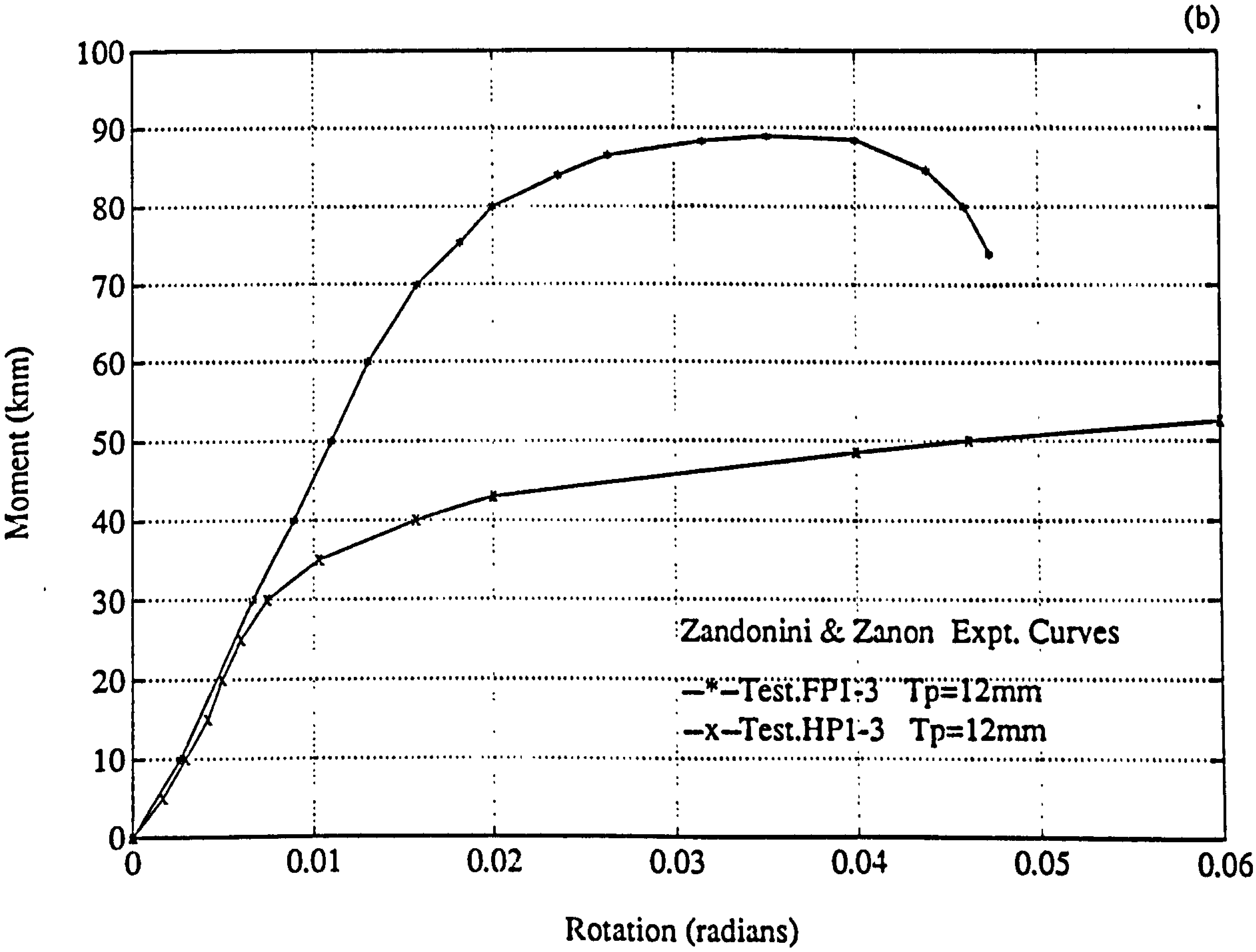


FIG.3-3 Effect of end-plate thickness on moment-rotation characteristics

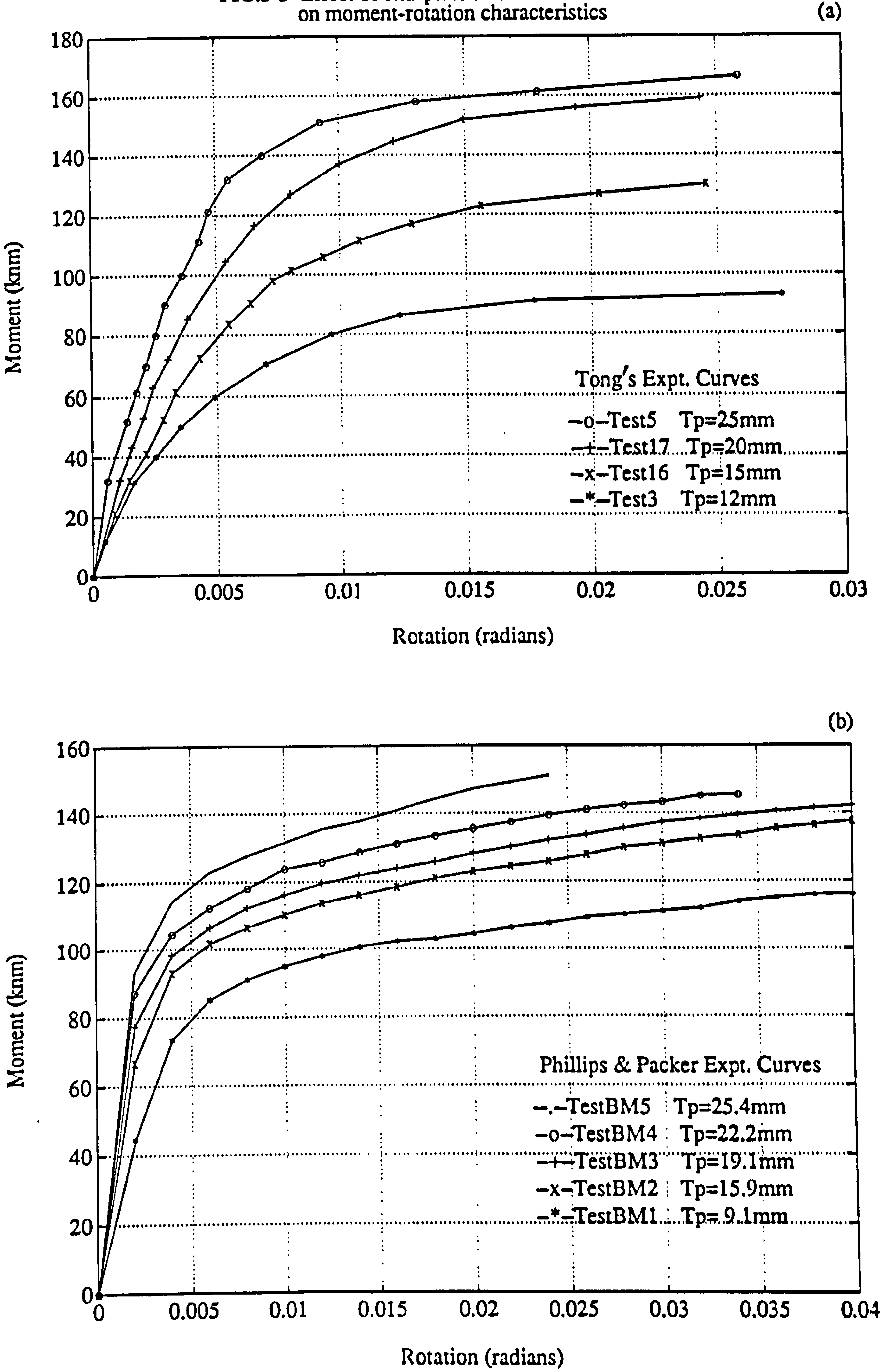
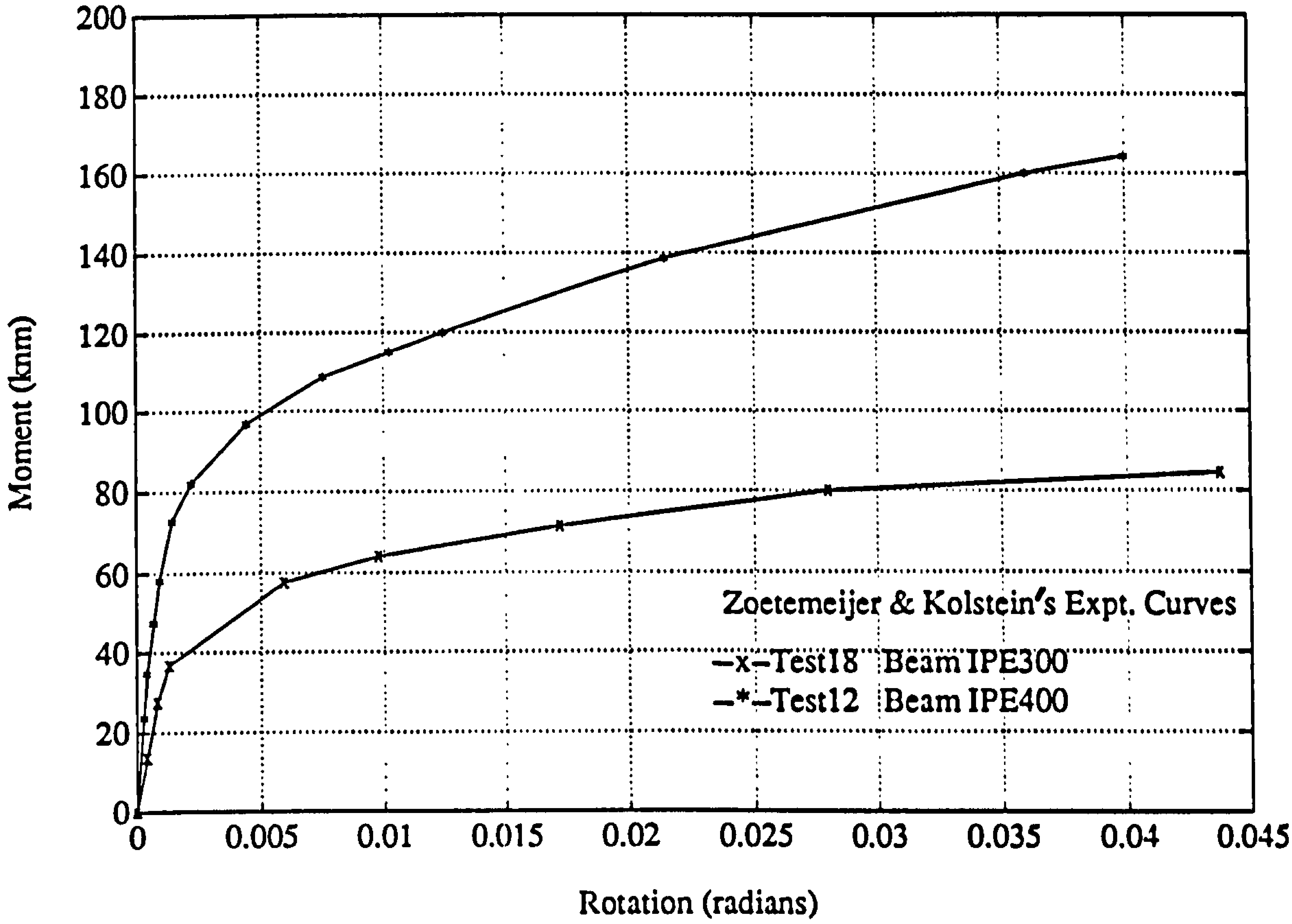


FIG.3-4 Effect of beam depth on moment-rotation characteristics

(a)



(b)

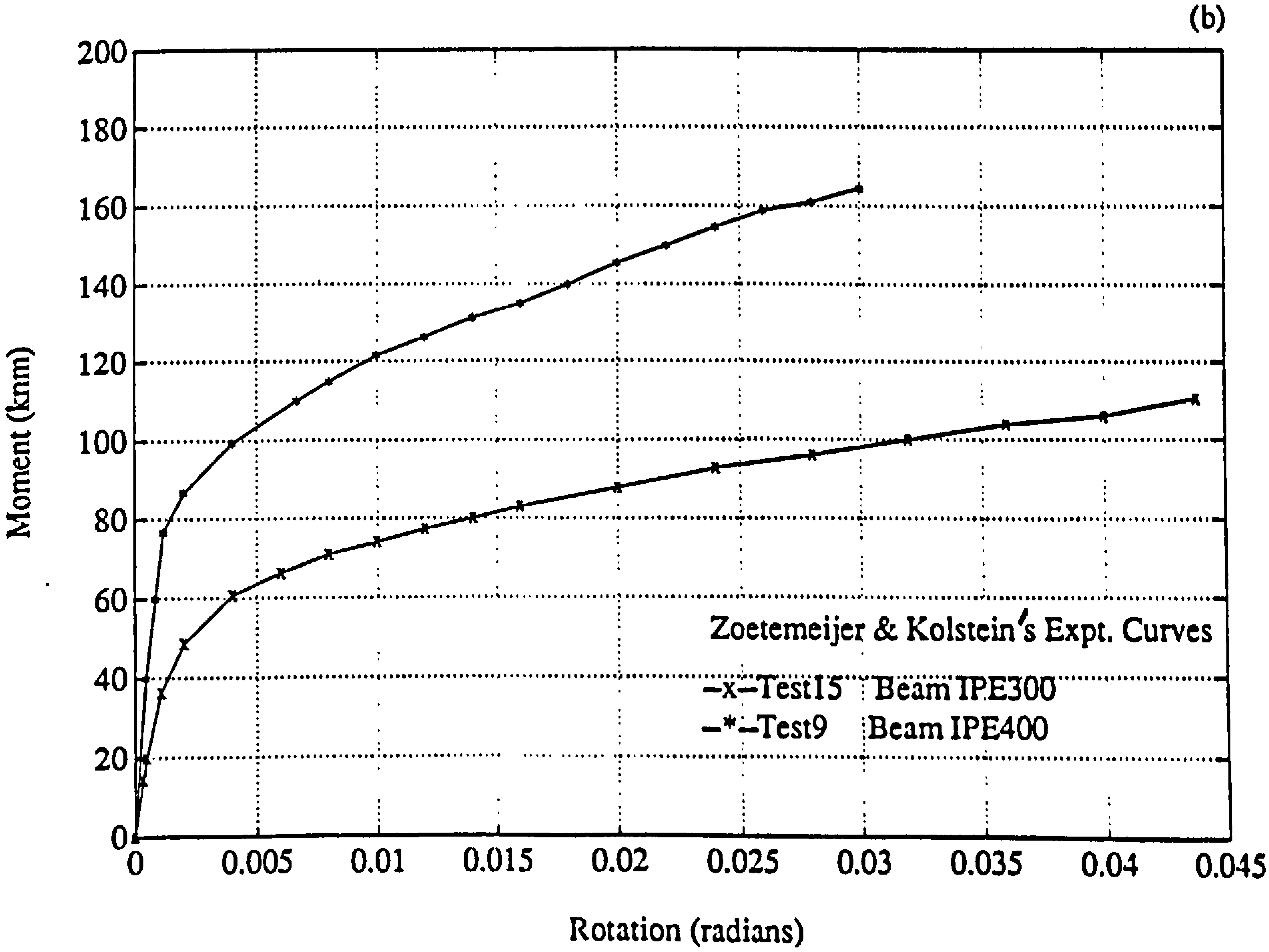
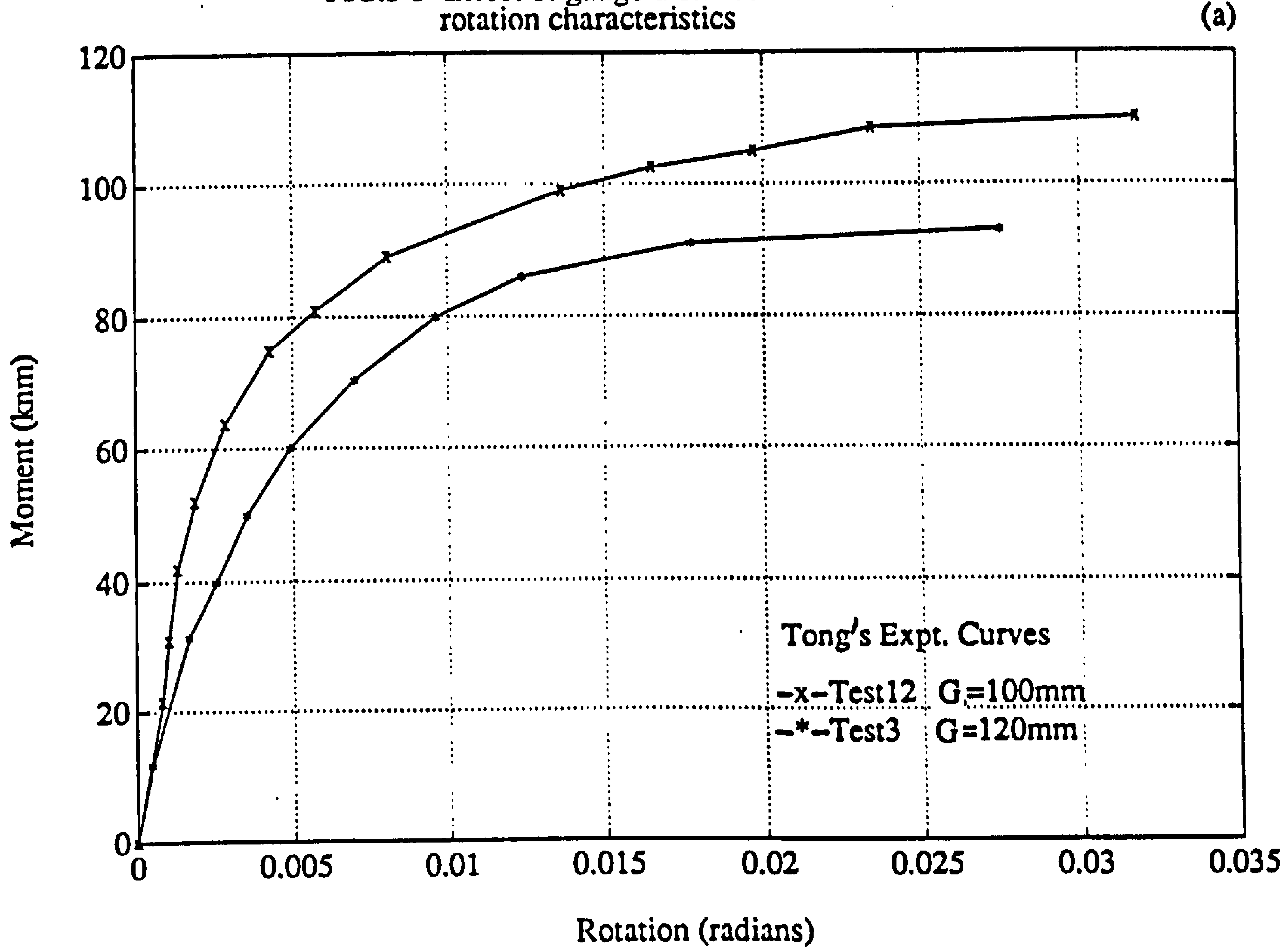


FIG.3-5 Effect of gauge distance on moment-rotation characteristics



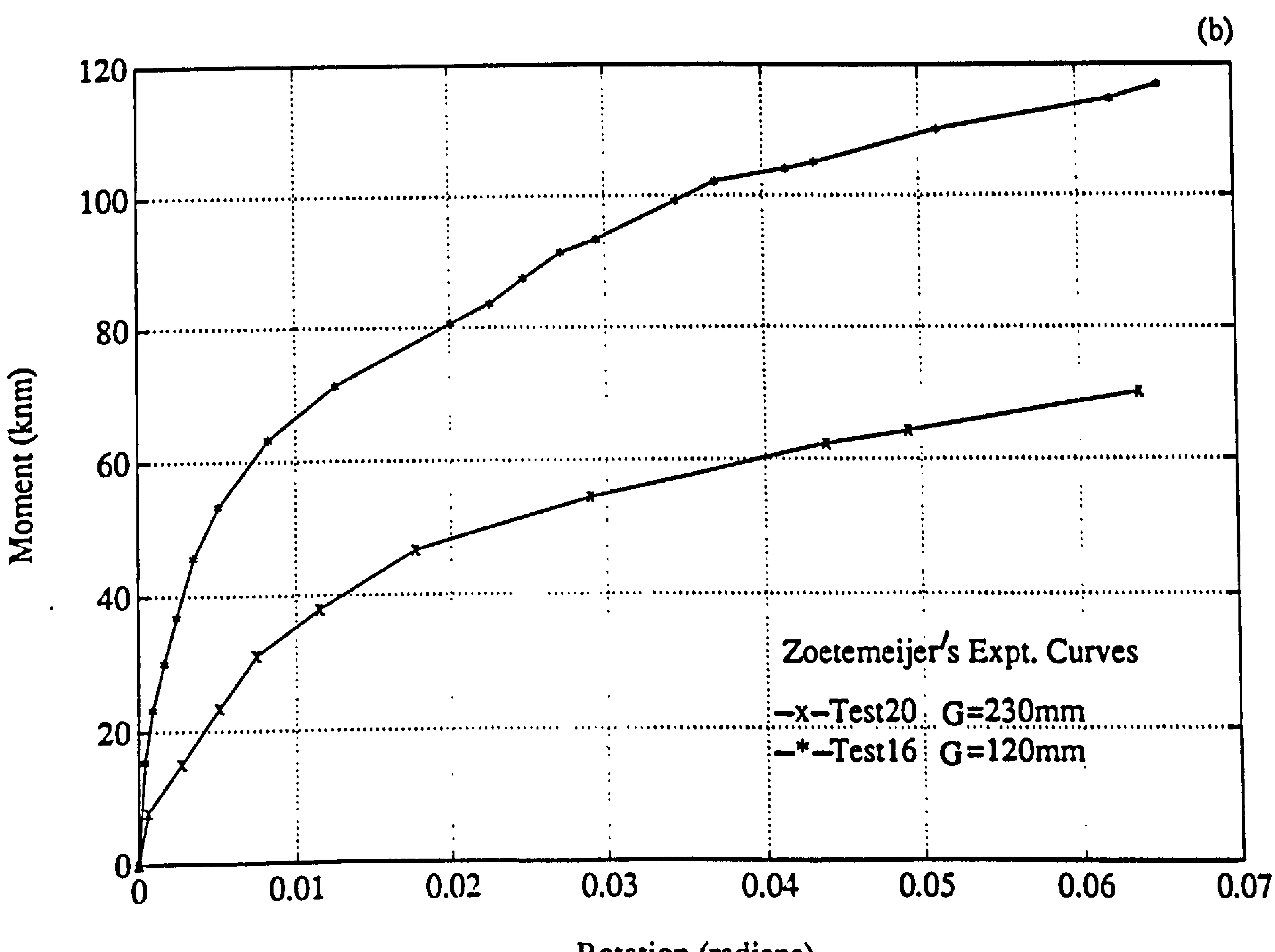
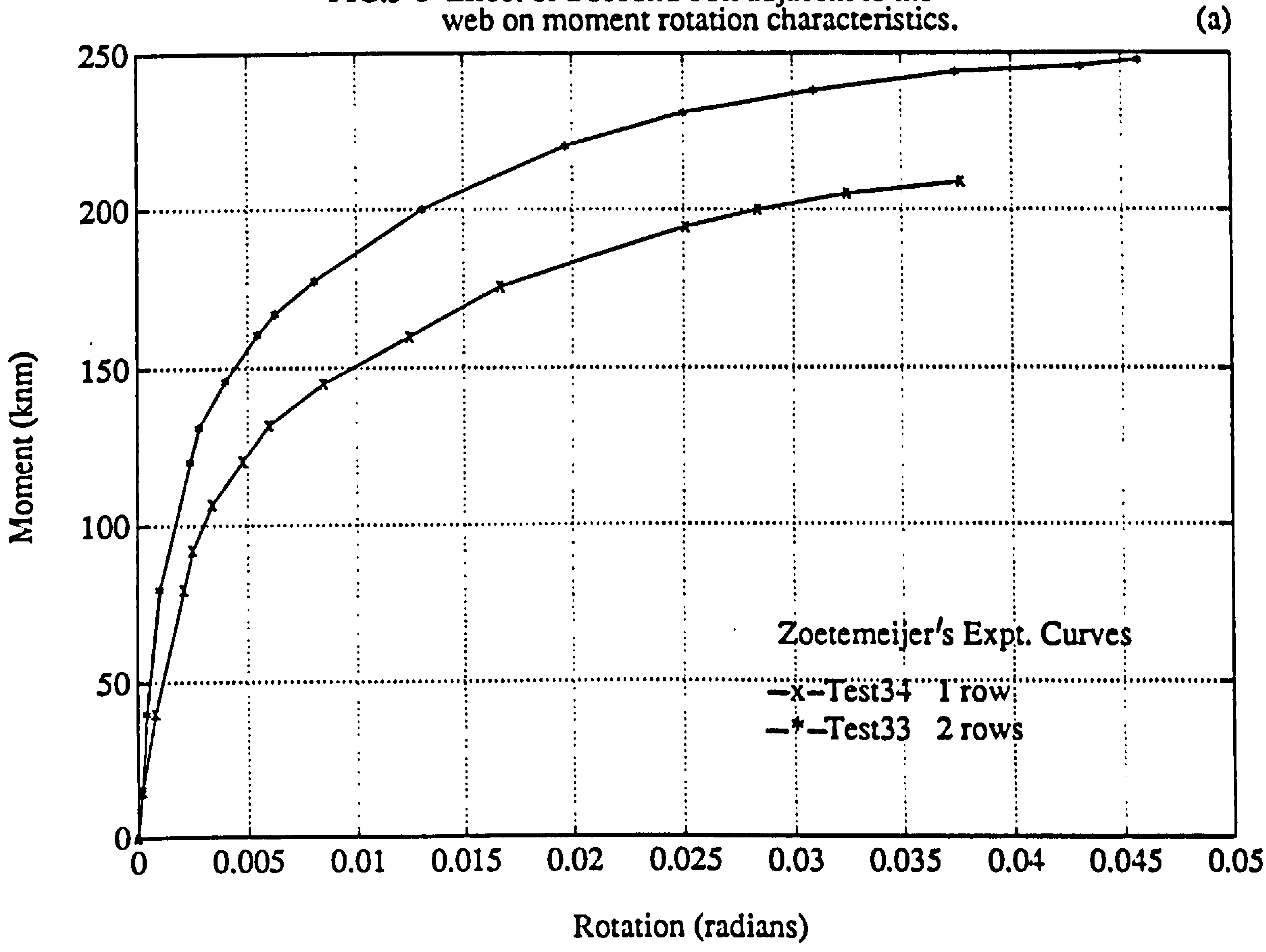


FIG.3-6 Effect of a second bolt adjacent to the web on moment rotation characteristics.



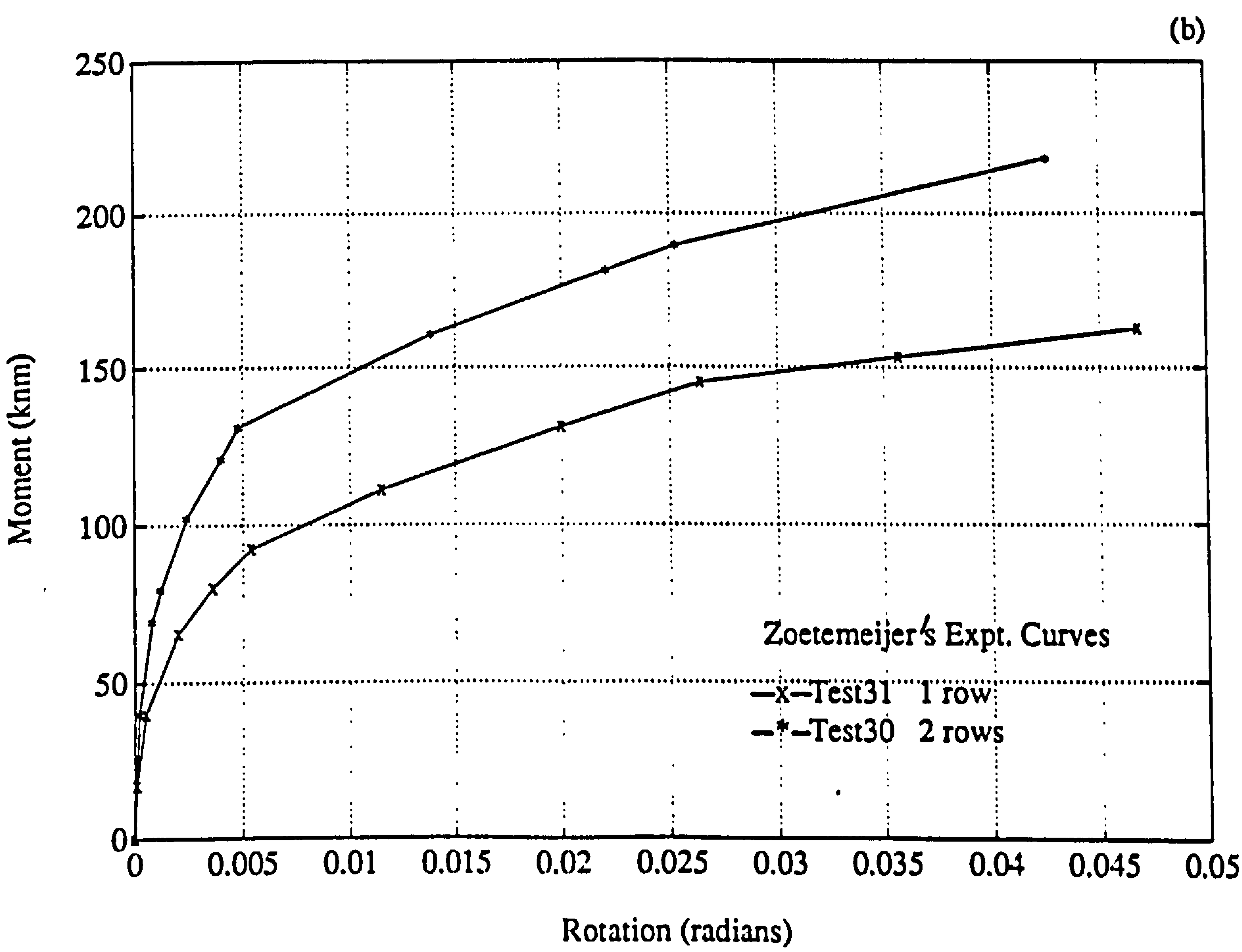
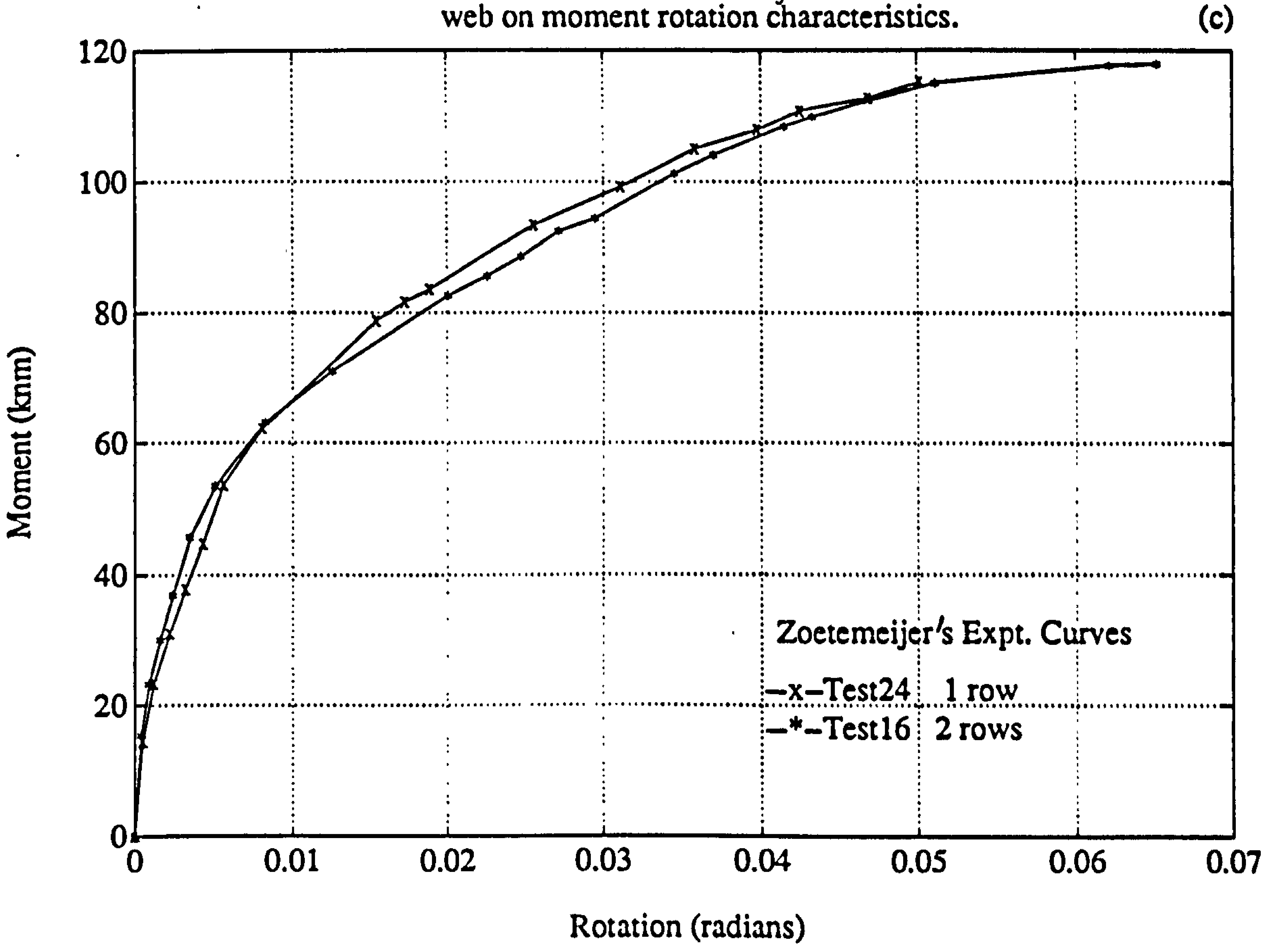


FIG.3-6 Effect of a second bolt adjacent to the web on moment rotation characteristics.



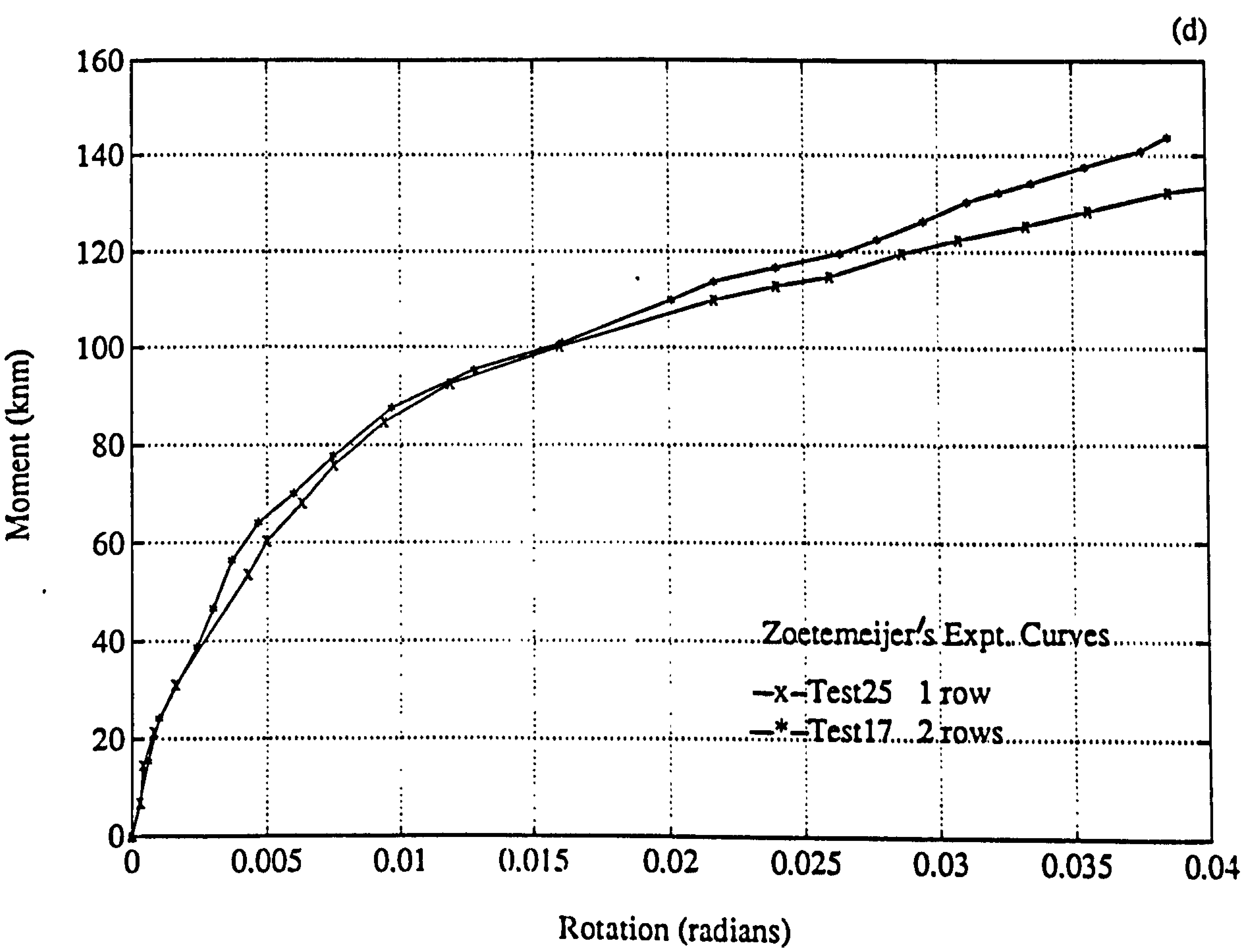
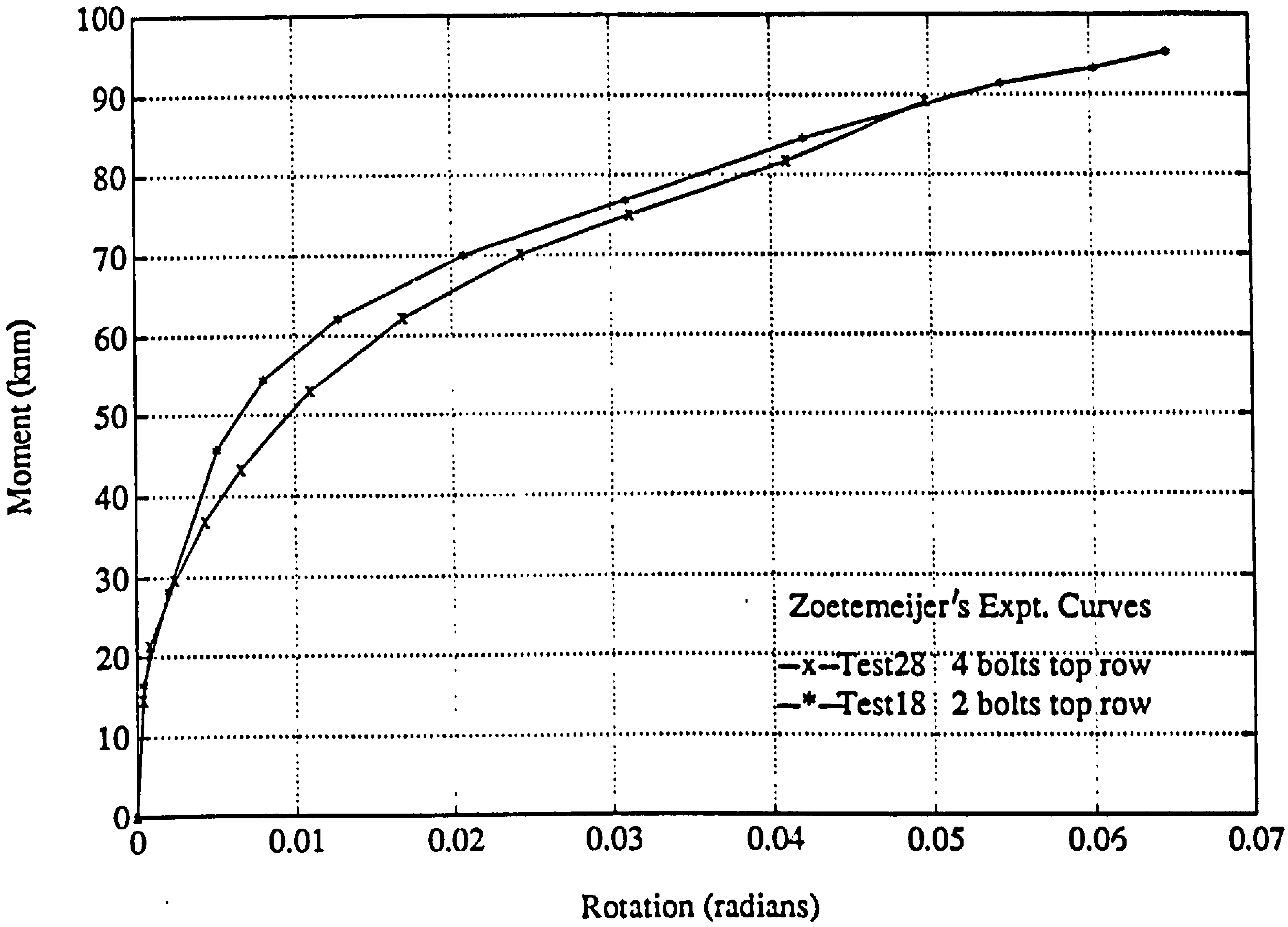


FIG.3-7 Effect of a second bolt adjacent to the flange on moment-rotation characteristics

(a)



(b)

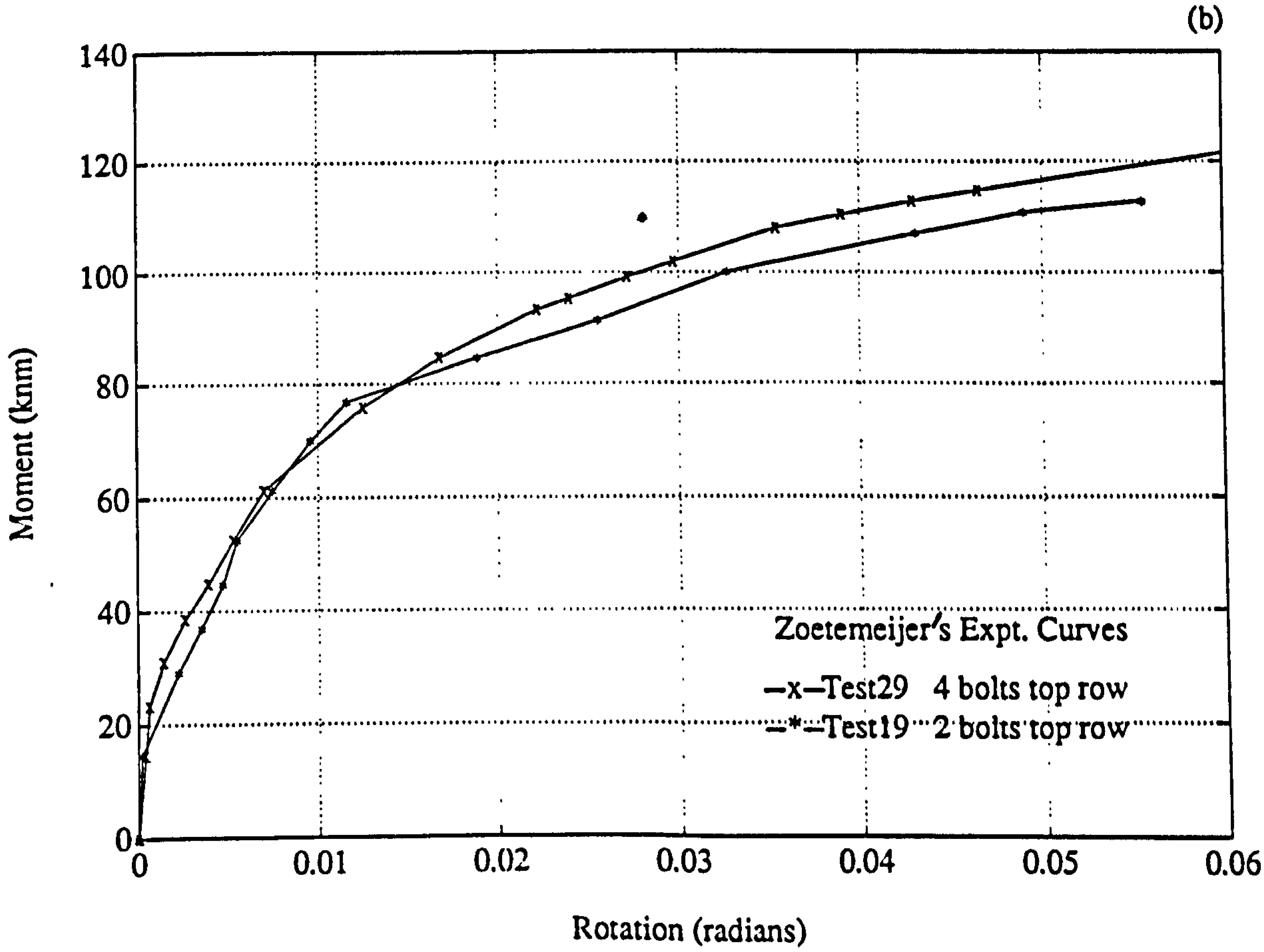
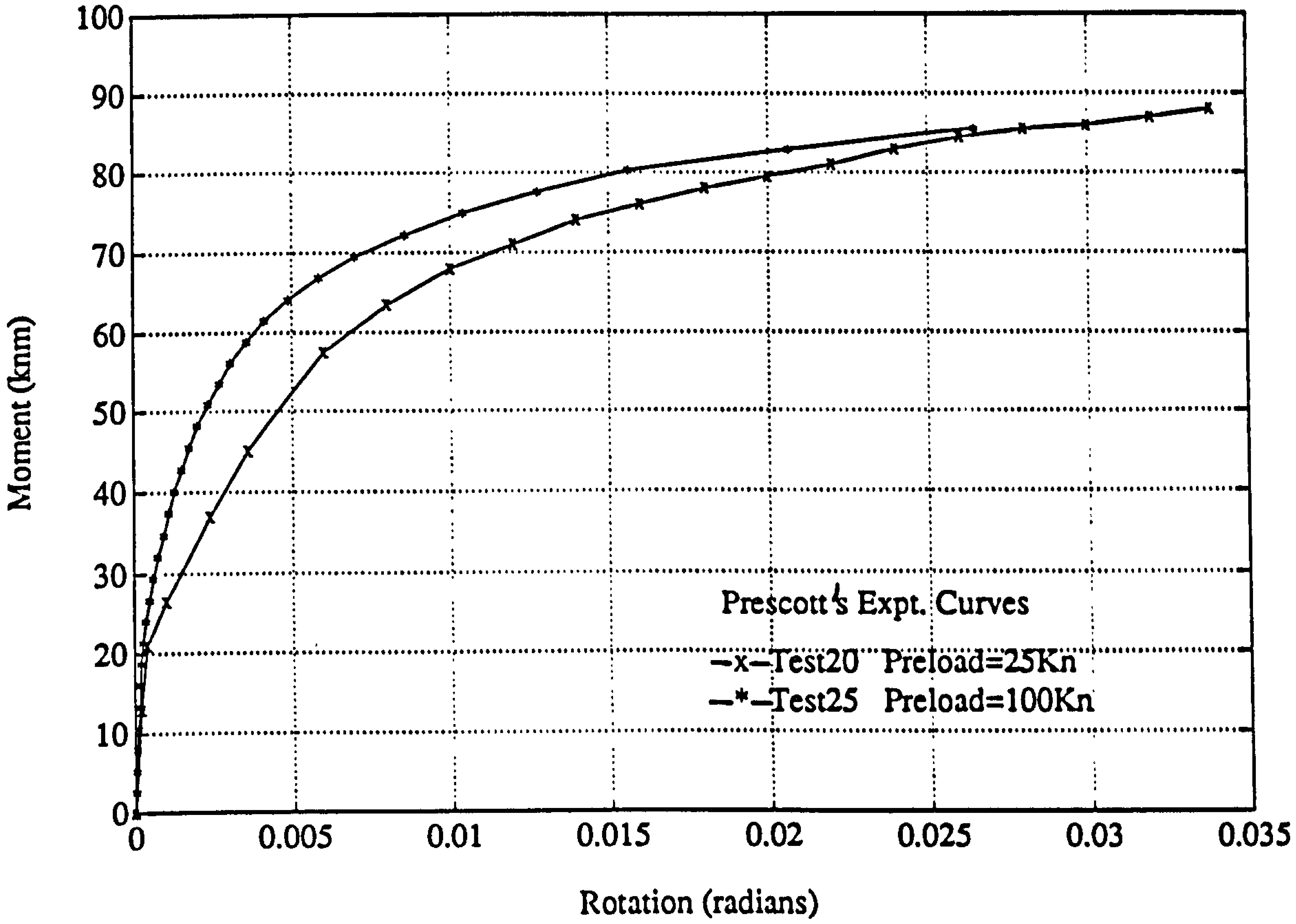


FIG.3-8 Effect of bolt preload on moment-rotation characteristics

(a)



(b)

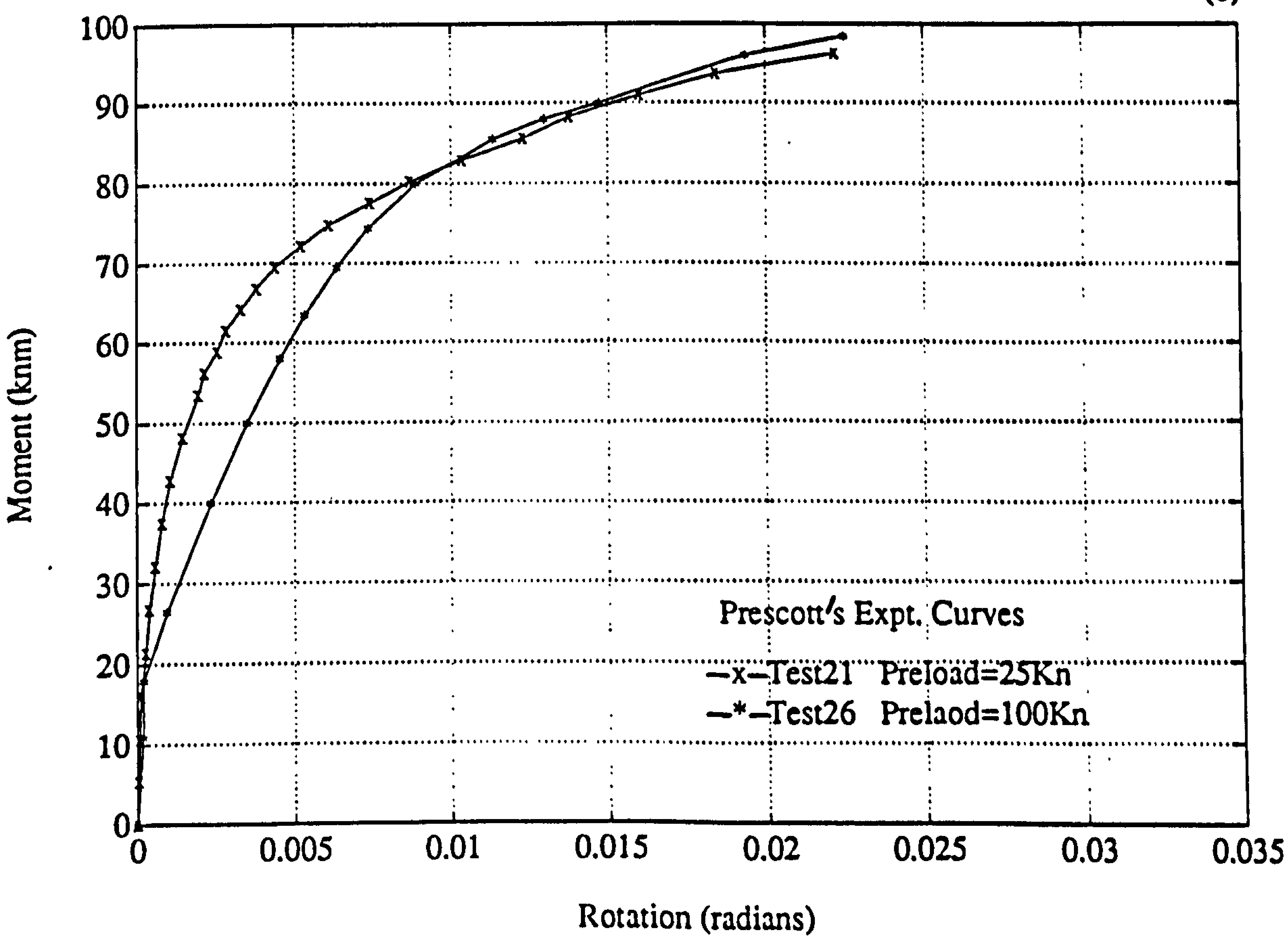
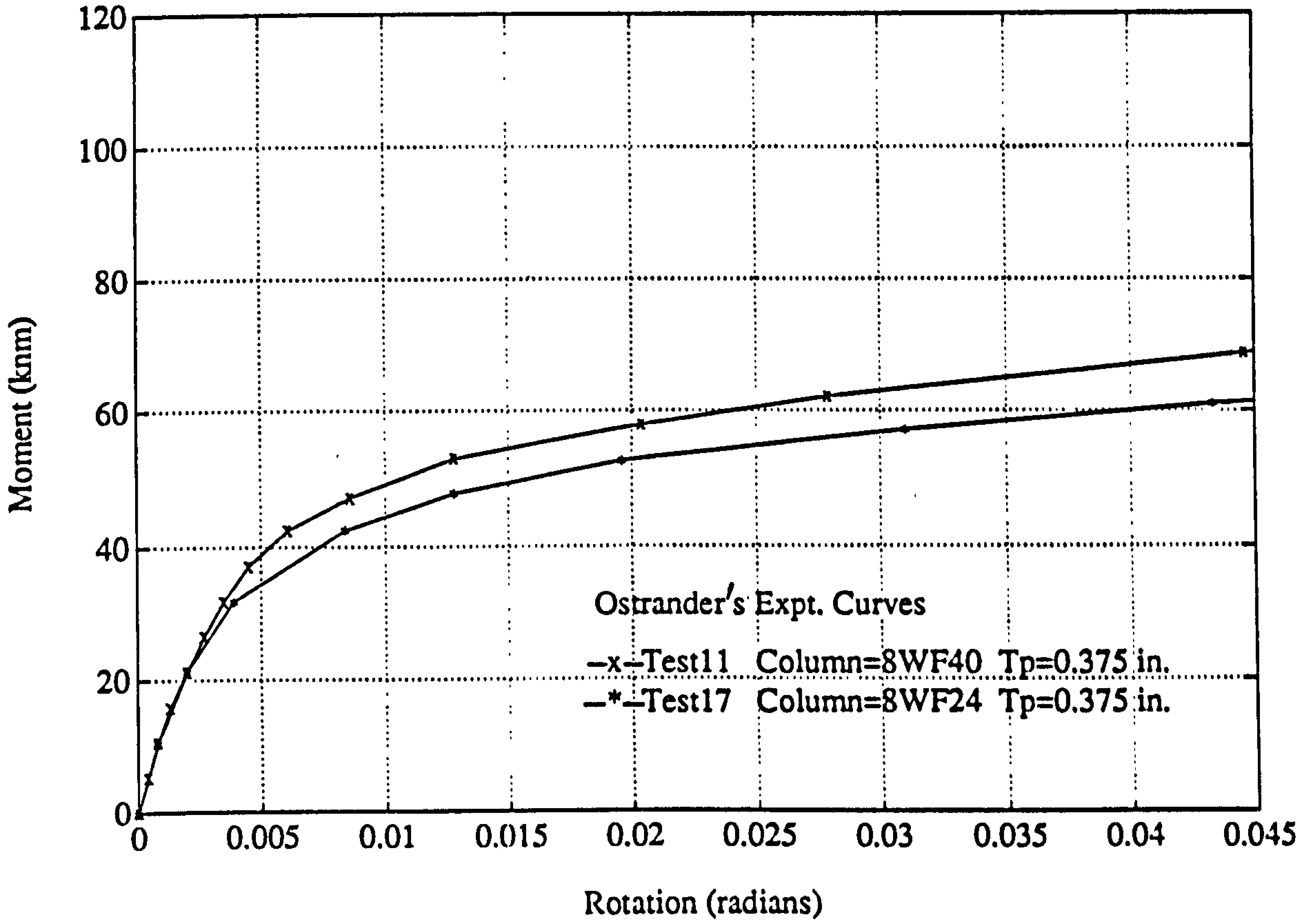


FIG.3-9 Effect of column section on moment-rotation characteristics

(a)



(b)

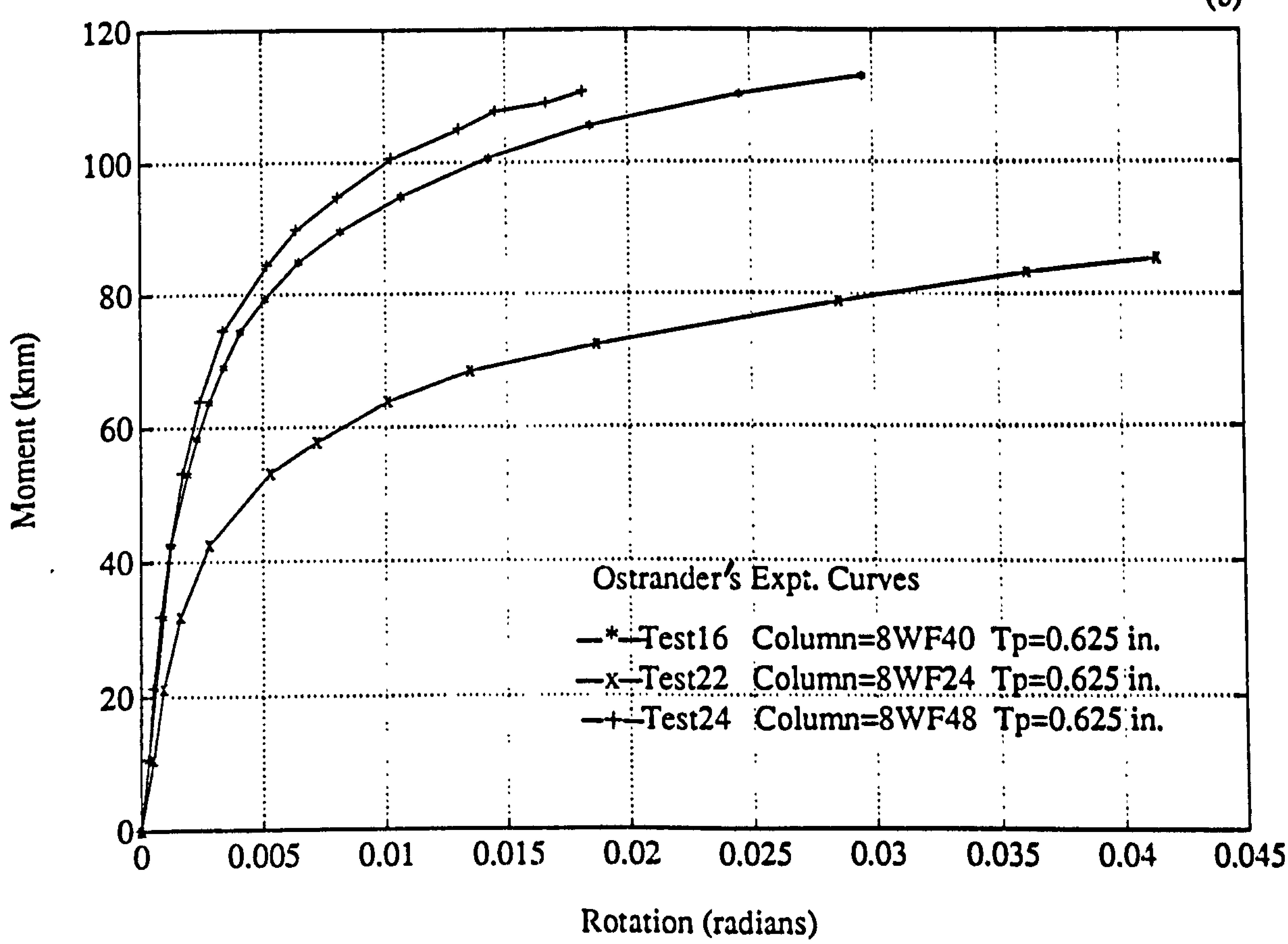


FIG.3-10 Effect of column web stiffeners on moment-rotation characteristics,

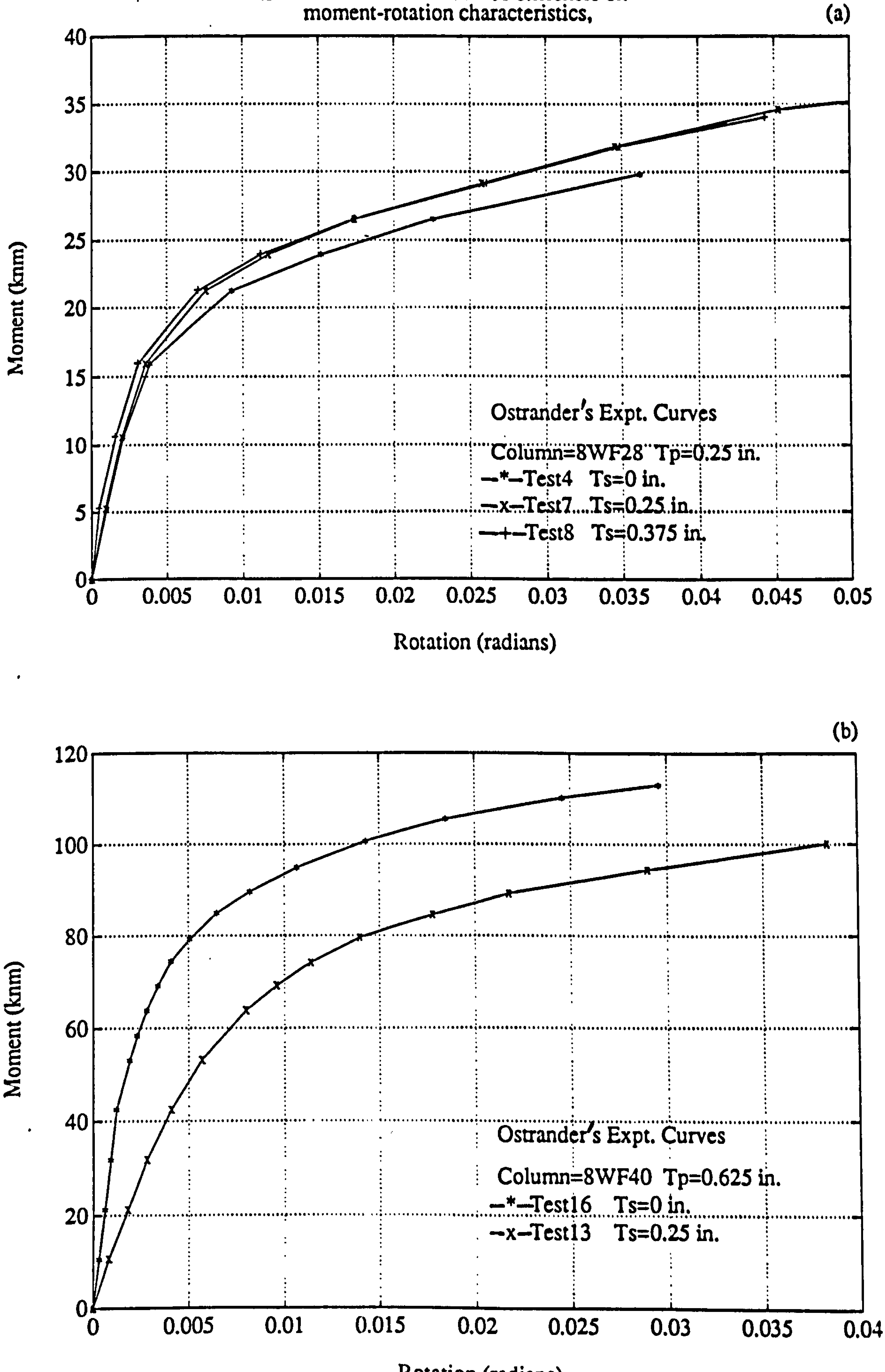
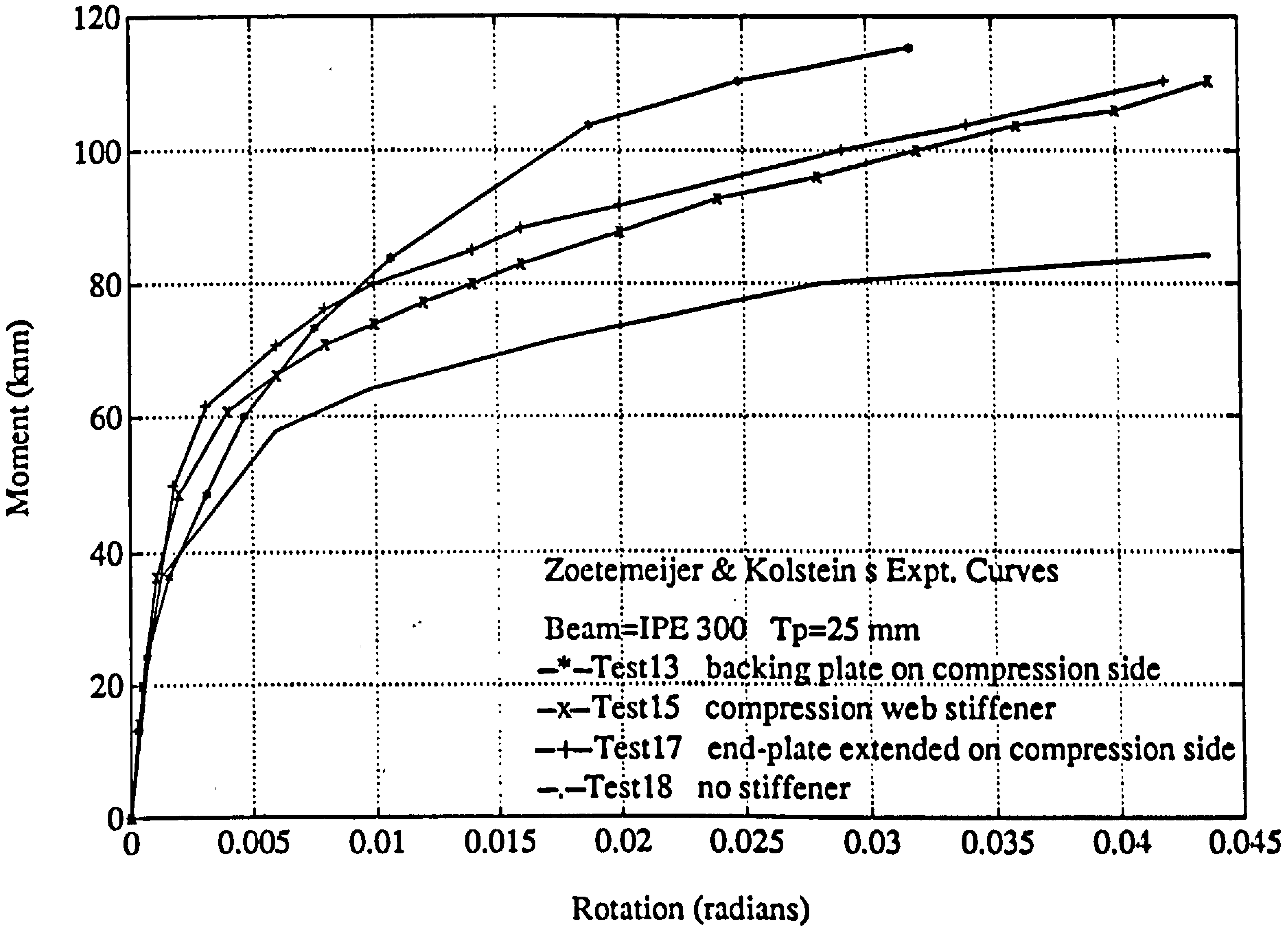
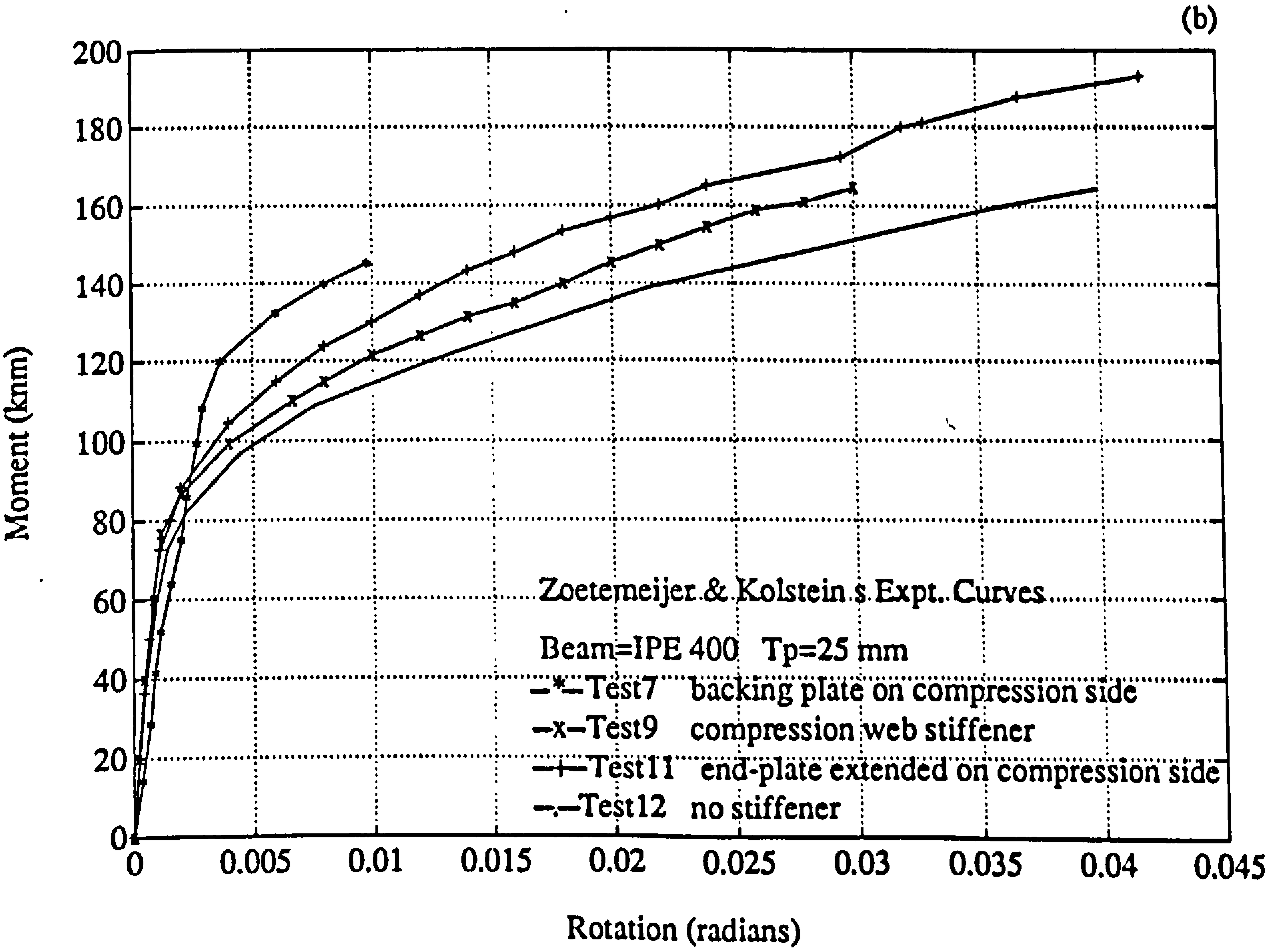


FIG.3-11 Effect of different types of column stiffening on moment-rotation characteristics.

(a)



(b)



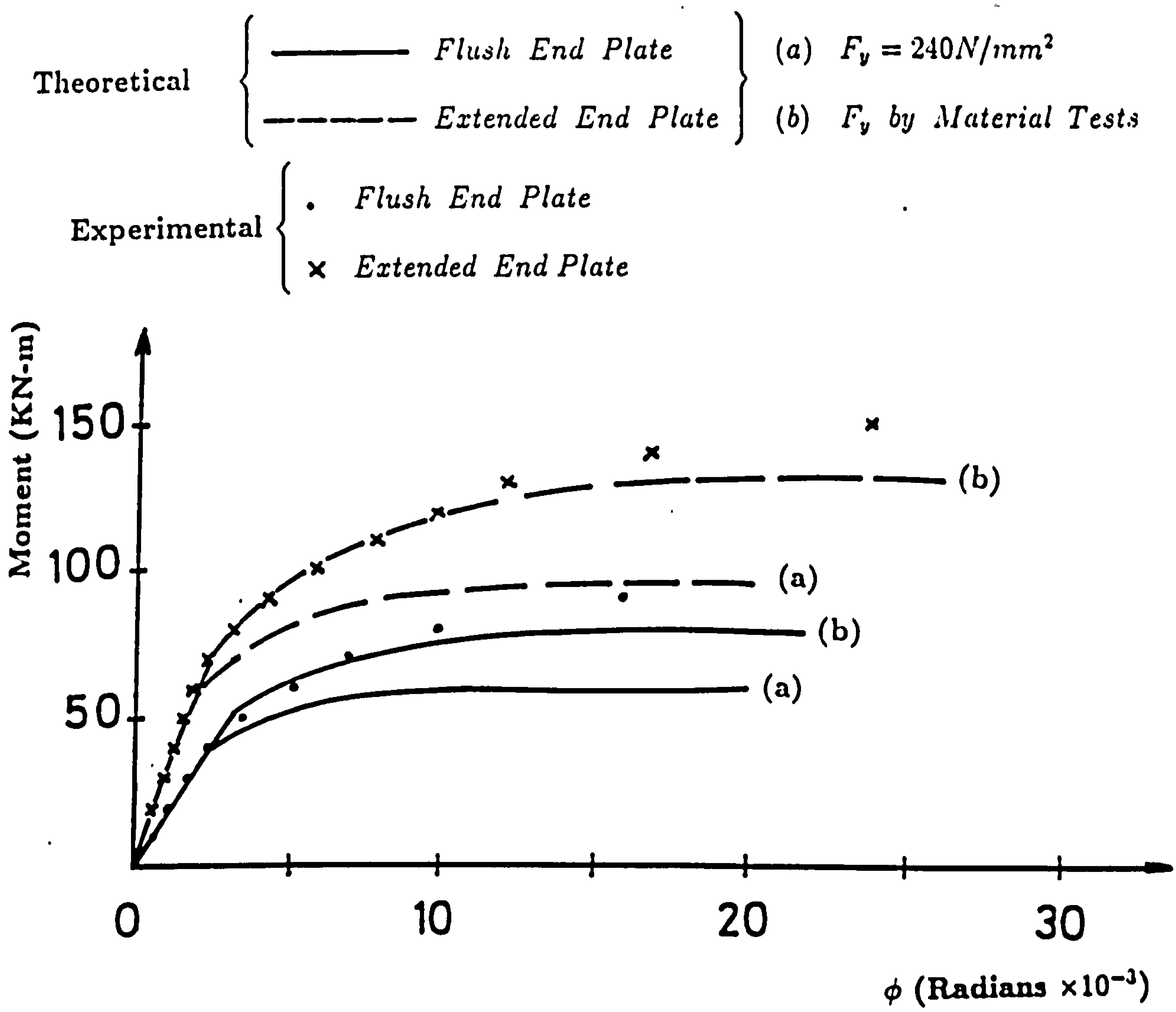


Figure 3.12 Effect of actual yield stress on moment-rotation characteristics [3.10].

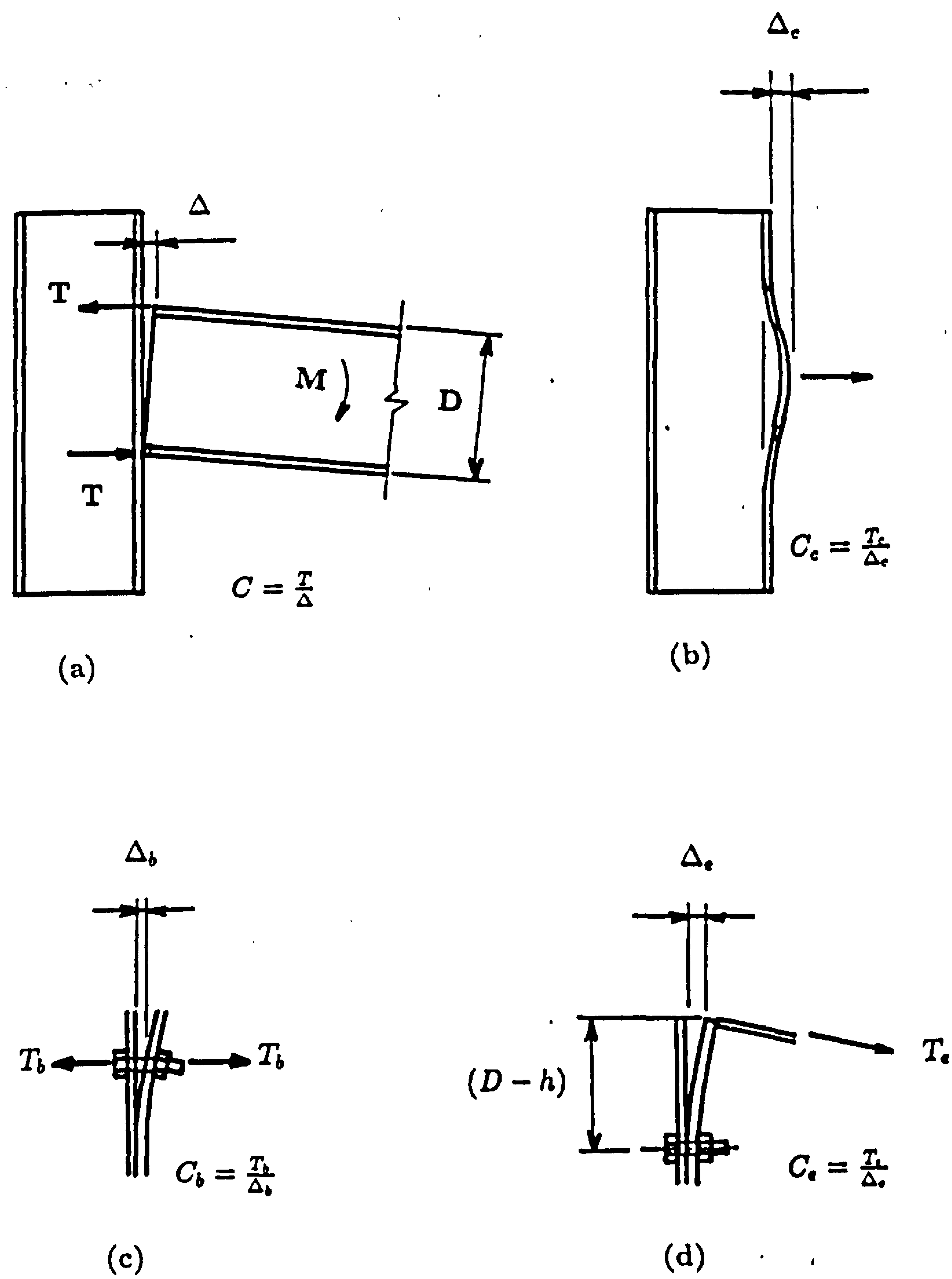
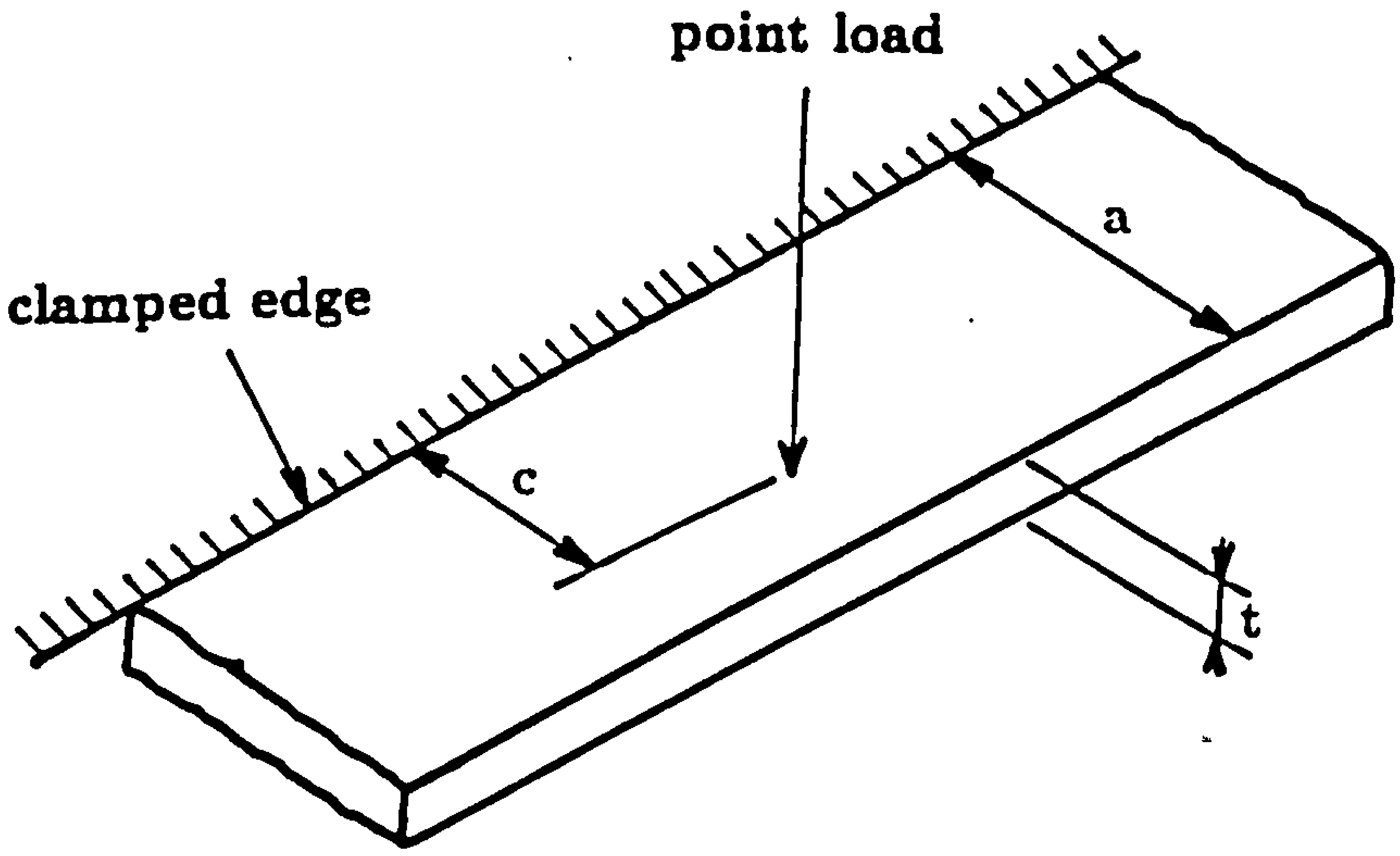
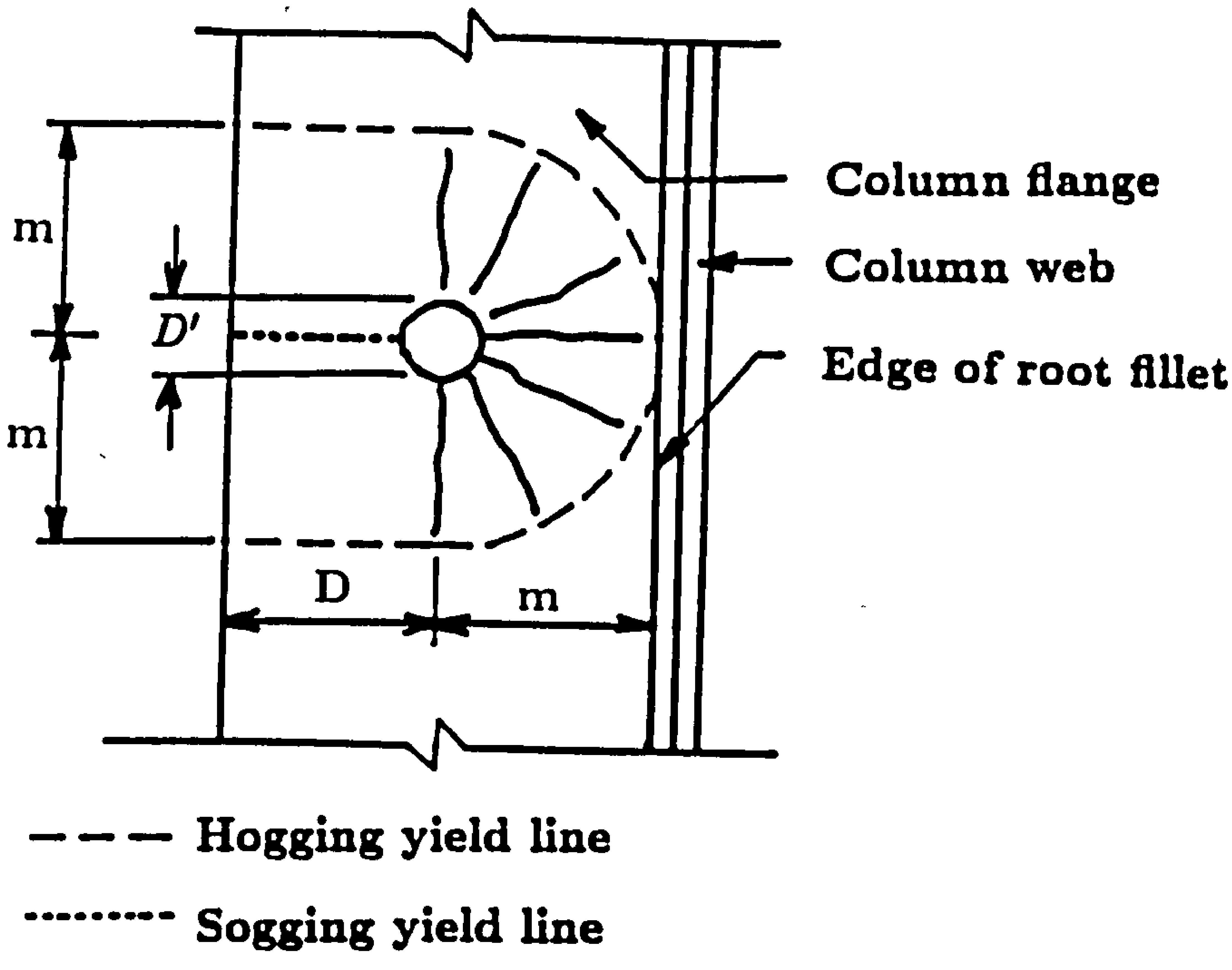


Figure 3.13 Deformation and associated stiffness of the flush end-plate connection assumed by Johnson and Law: (a) Overall idealization, (b) Column flange idealization, (c) Bolt idealization, and (d) End-plate idealization [3.31].

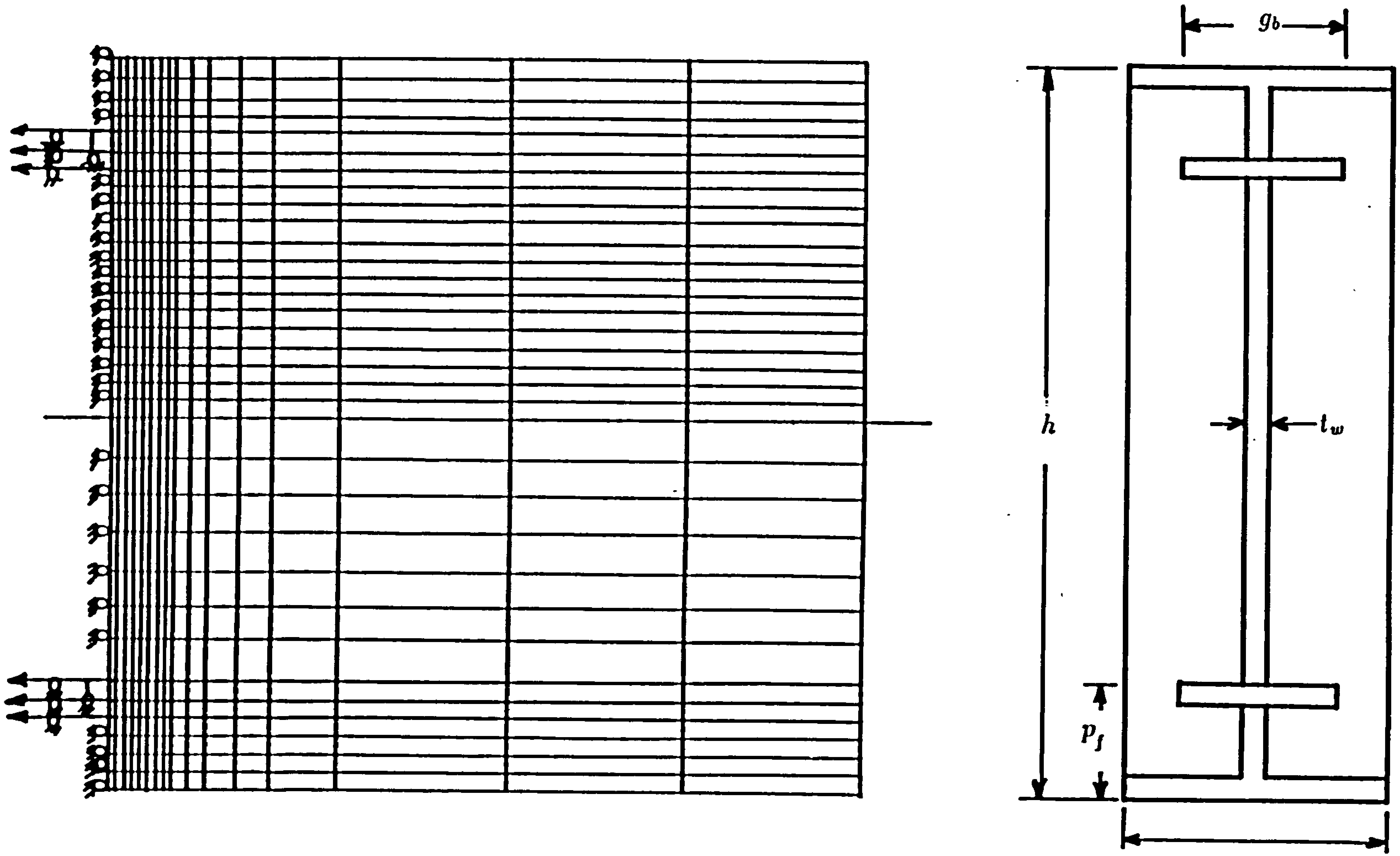


(a) Idealisation of column flange



(b) Assumed yield line pattern of column flange

Figure 3.14 (a) Idealization of column flange (b) Assumed yield line pattern of column flange [3.31].



(a)

(b)

Figure 3.15 (a) Two dimensional finite element mesh configuration, (b) Section [3.34].

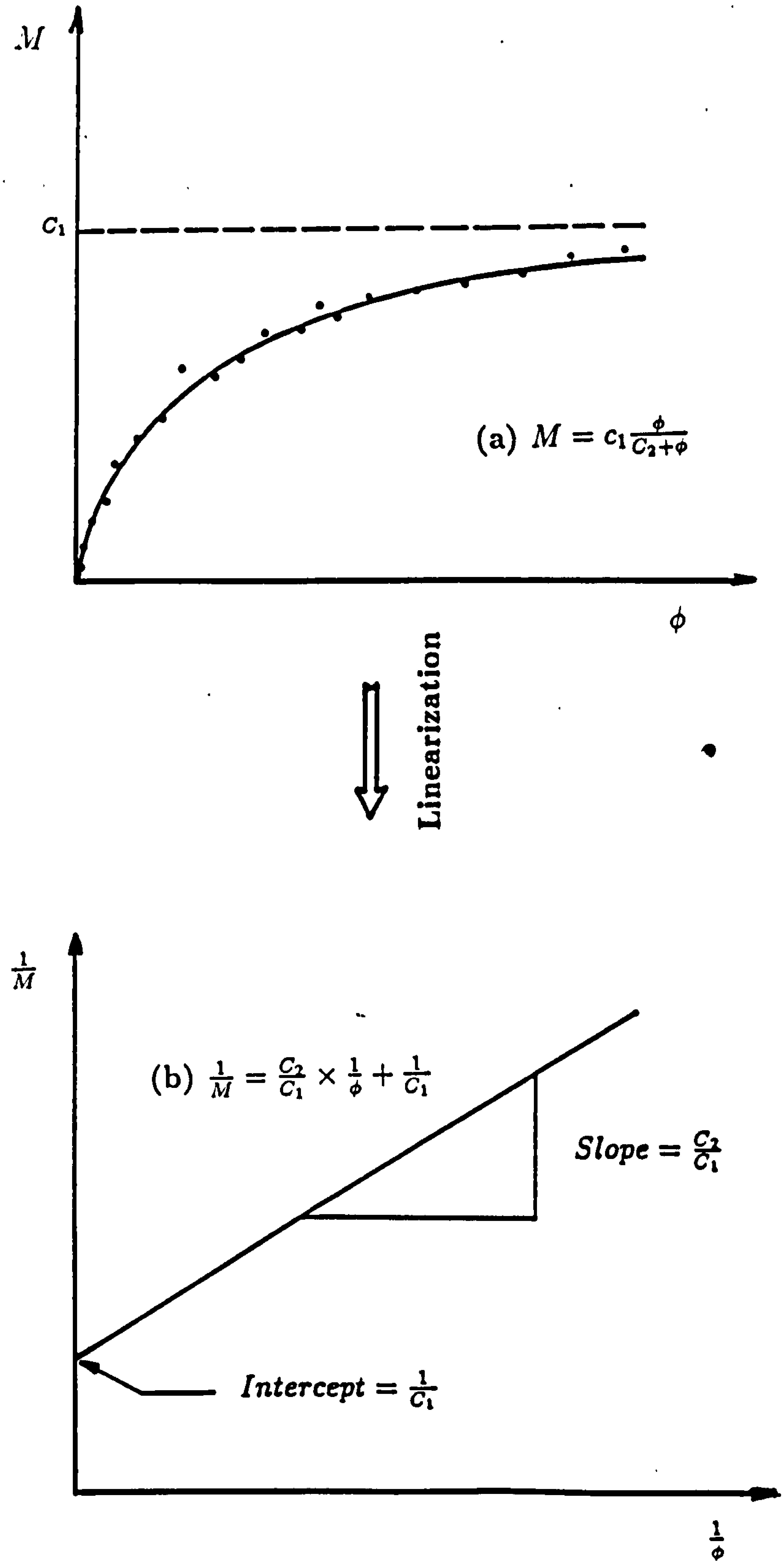


Figure 3.16 Modelling of moment-rotation relationships (a) Fit of a specific moment versus connection rotation, (b) Linearization version of the moment-rotation relationship.

FIG.3-17 Comparison between analytical moment-rotation relationships and Ostrander's test 1 (unstiffened column).

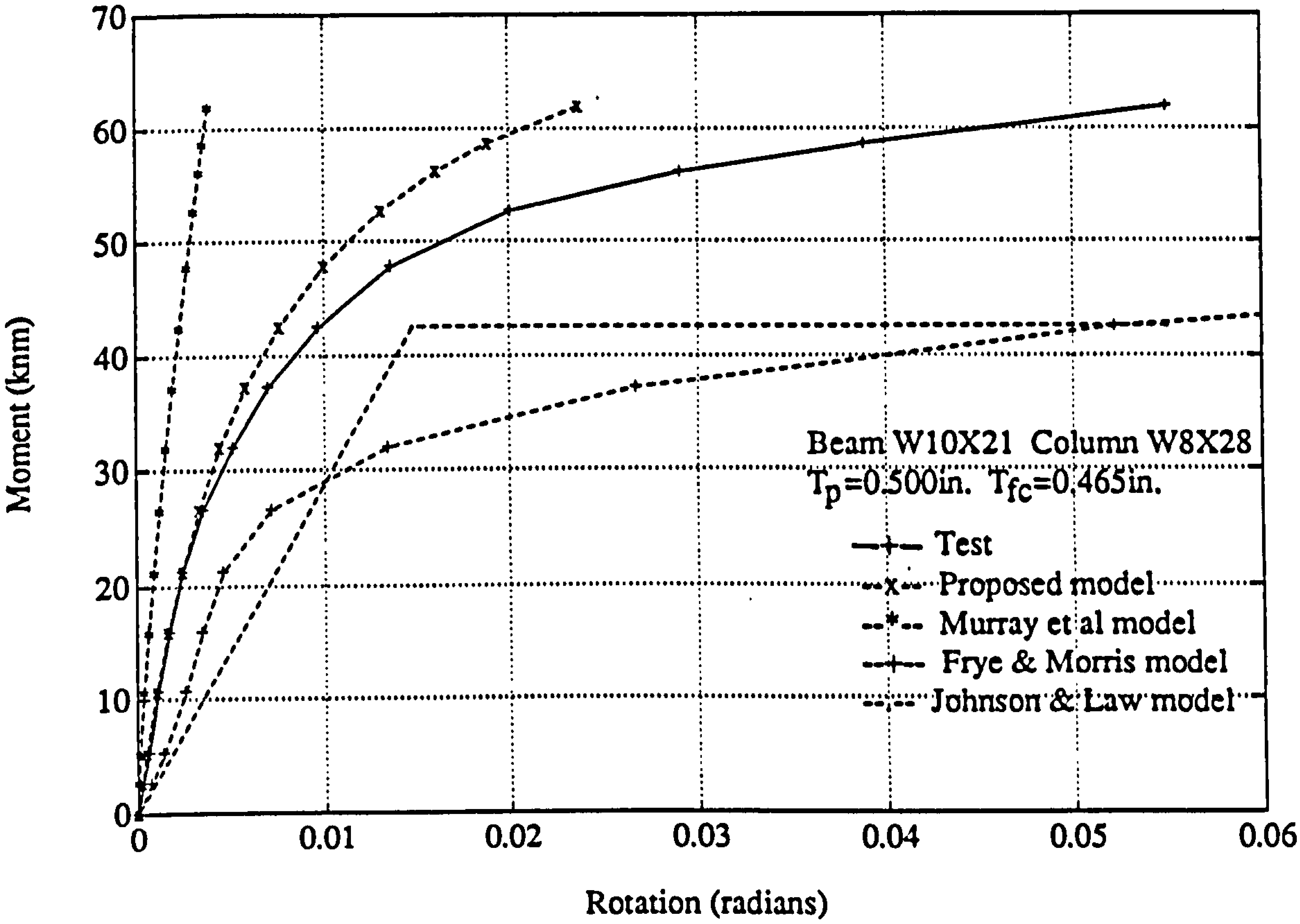


FIG.3-18 Comparison between analytical moment-rotation relationships and Ostrander's test 3 (unstiffened column).

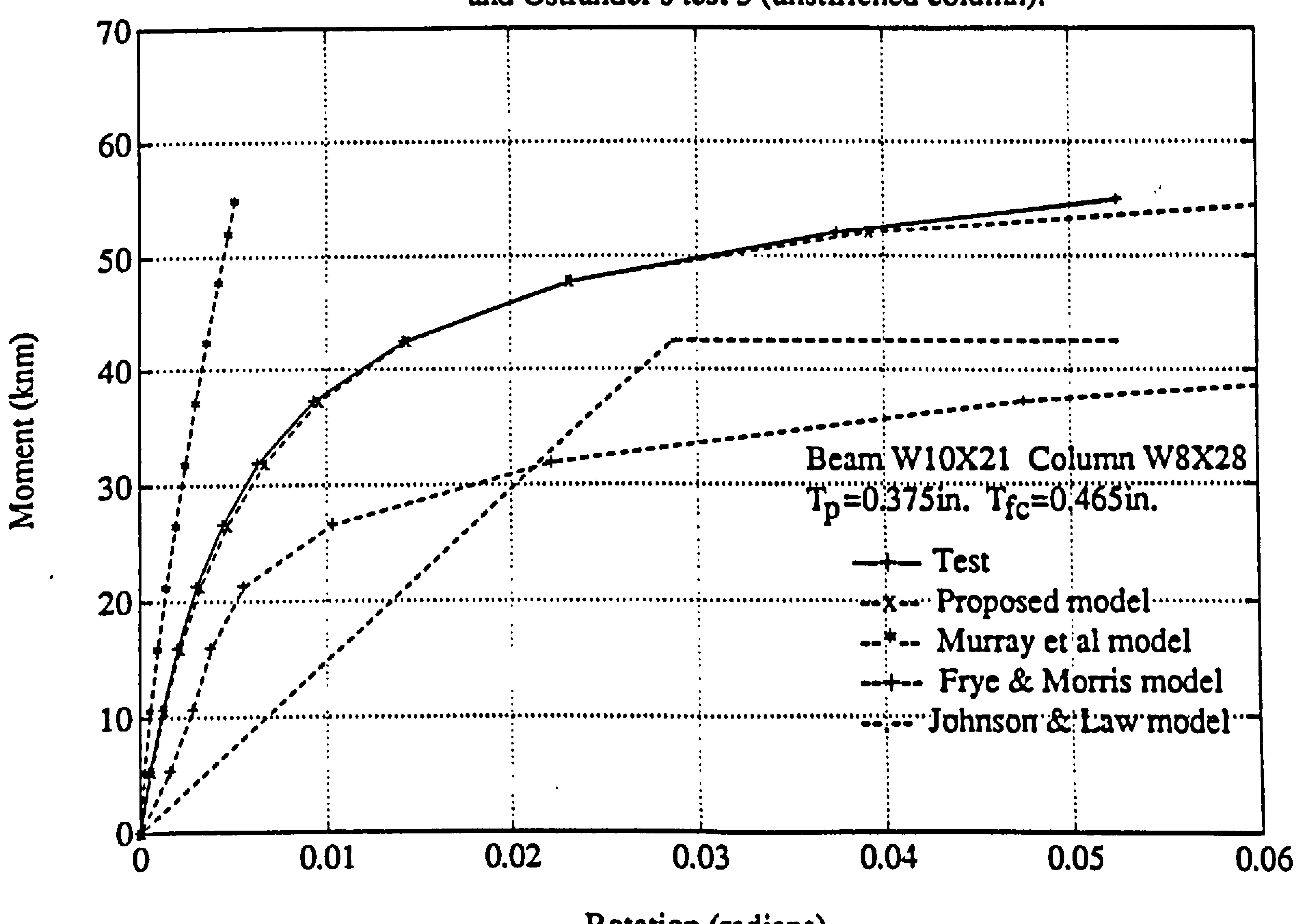


FIG.3-19 Comparison between analytical moment-rotation relationships and Ostrander's test 4 (unstiffened column).

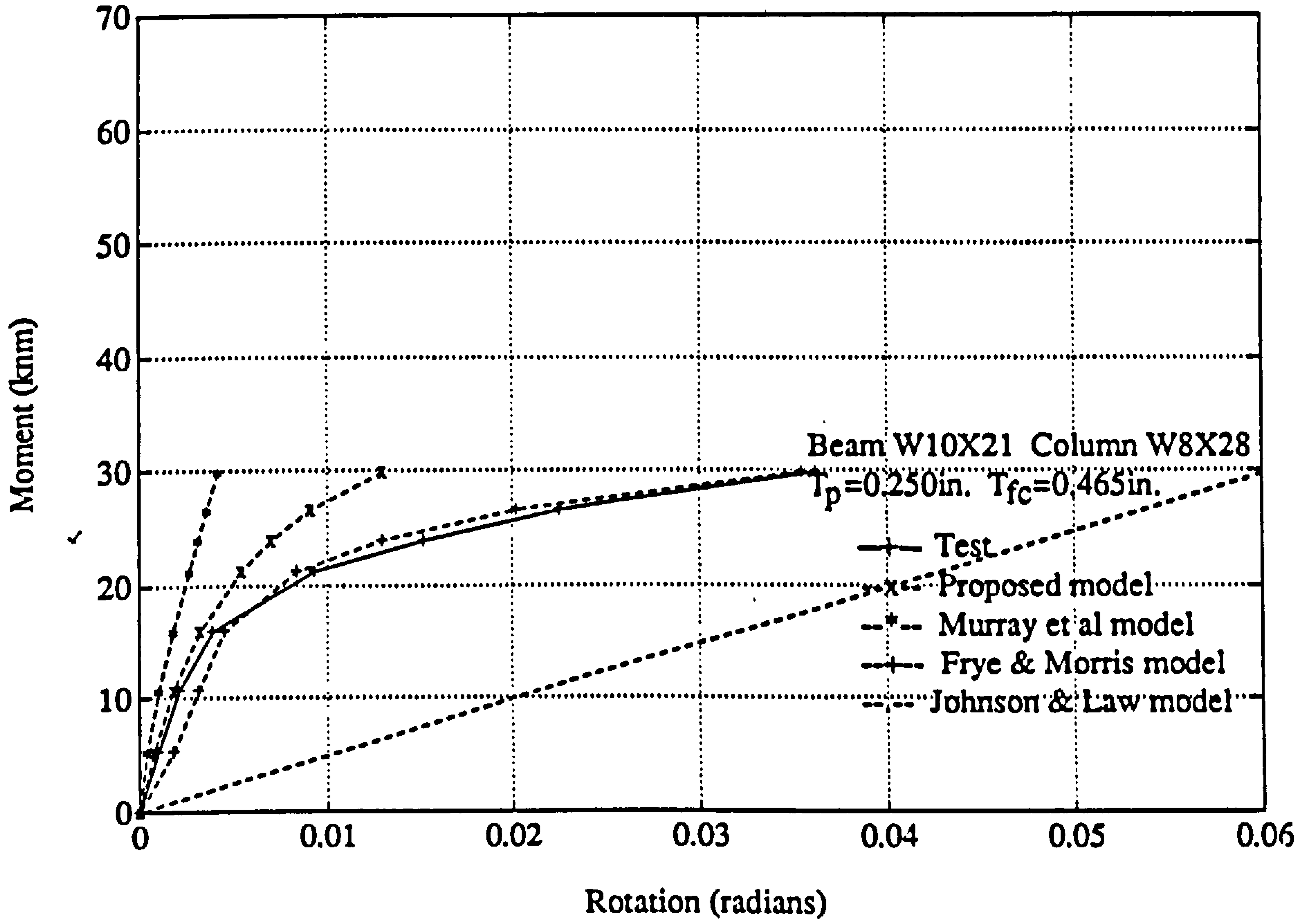


FIG.3-20 Comparison between analytical moment-rotation relationships and Ostrander's test 9 (unstiffened column).

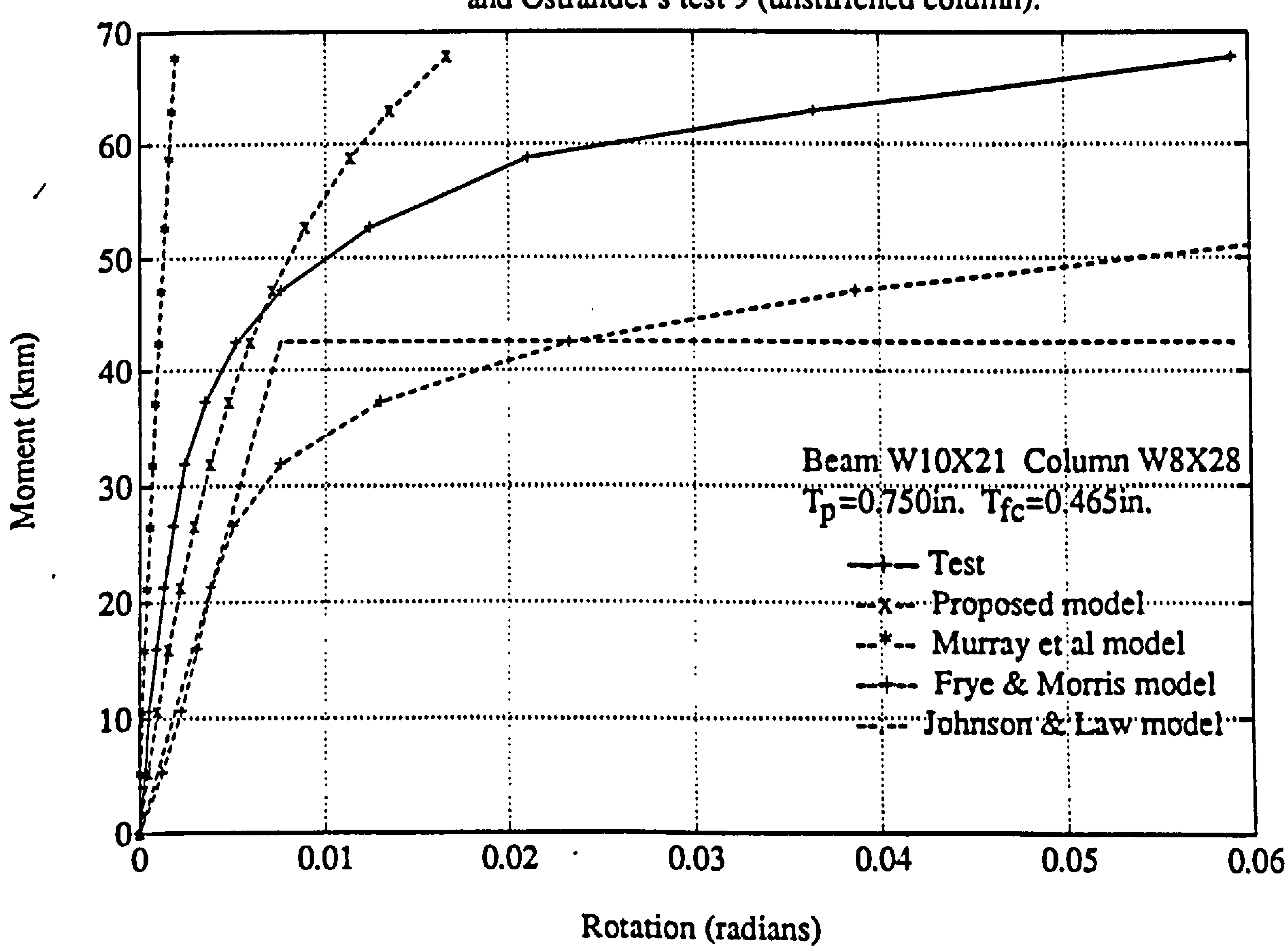


FIG.3-21 Comparison between analytical moment-rotation relationships and Ostrander's test 11 (unstiffened column).

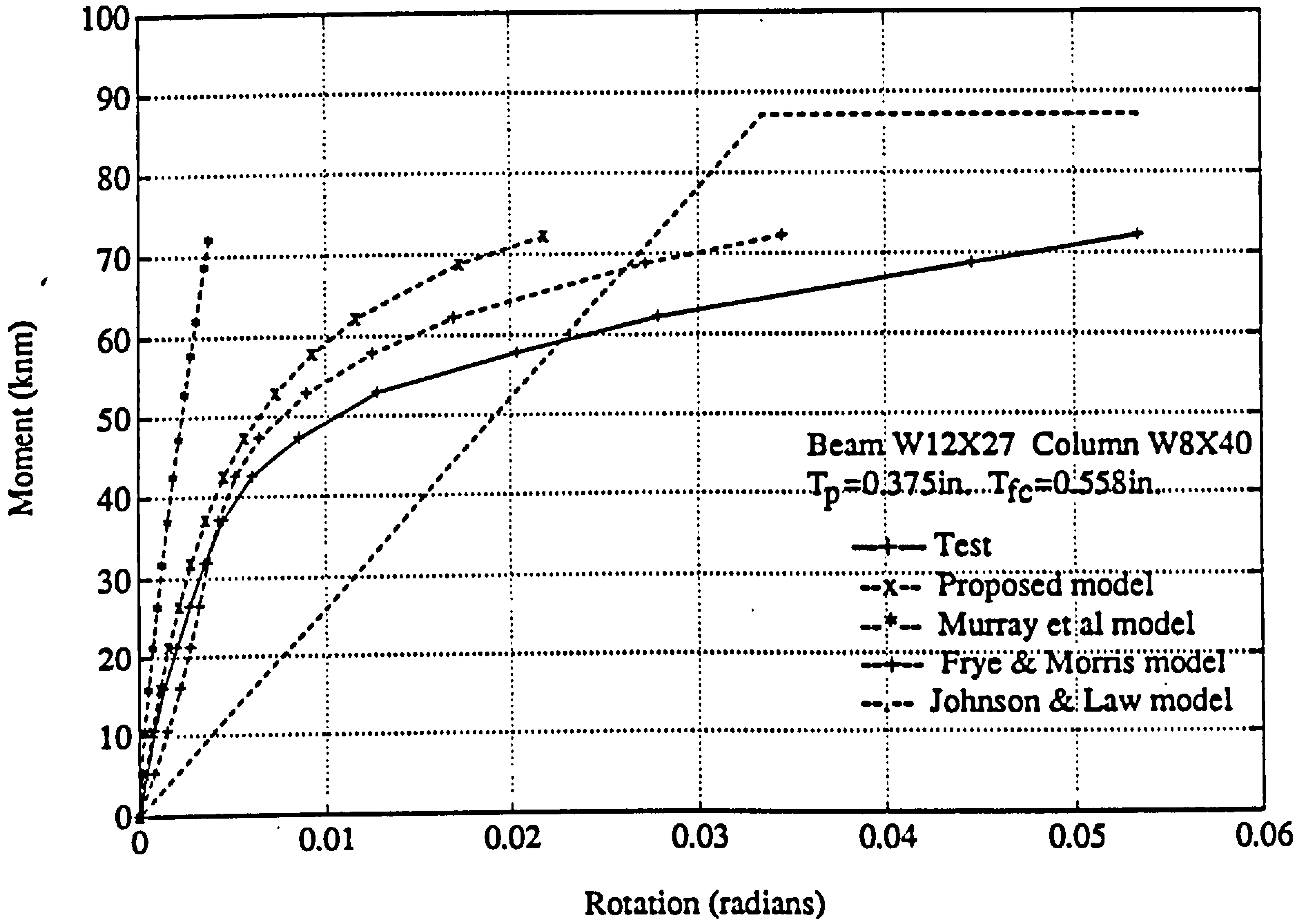


FIG.3-22 Comparison between analytical moment-rotation relationships and Ostrander's test 12 (unstiffened column).

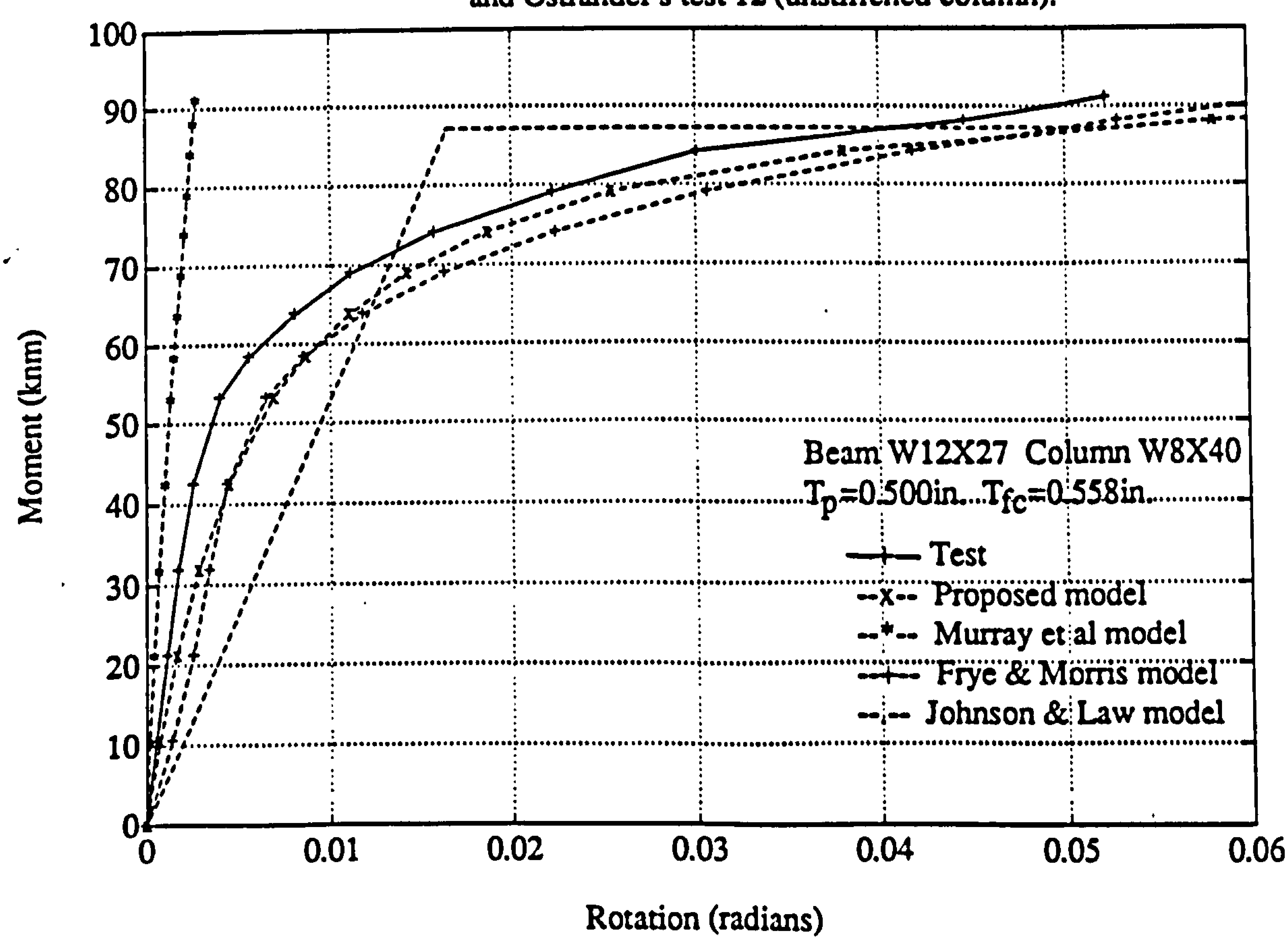


FIG.3-23 Comparison between analytical moment-rotation relationships and Ostrander's test 13 (unstiffened column).

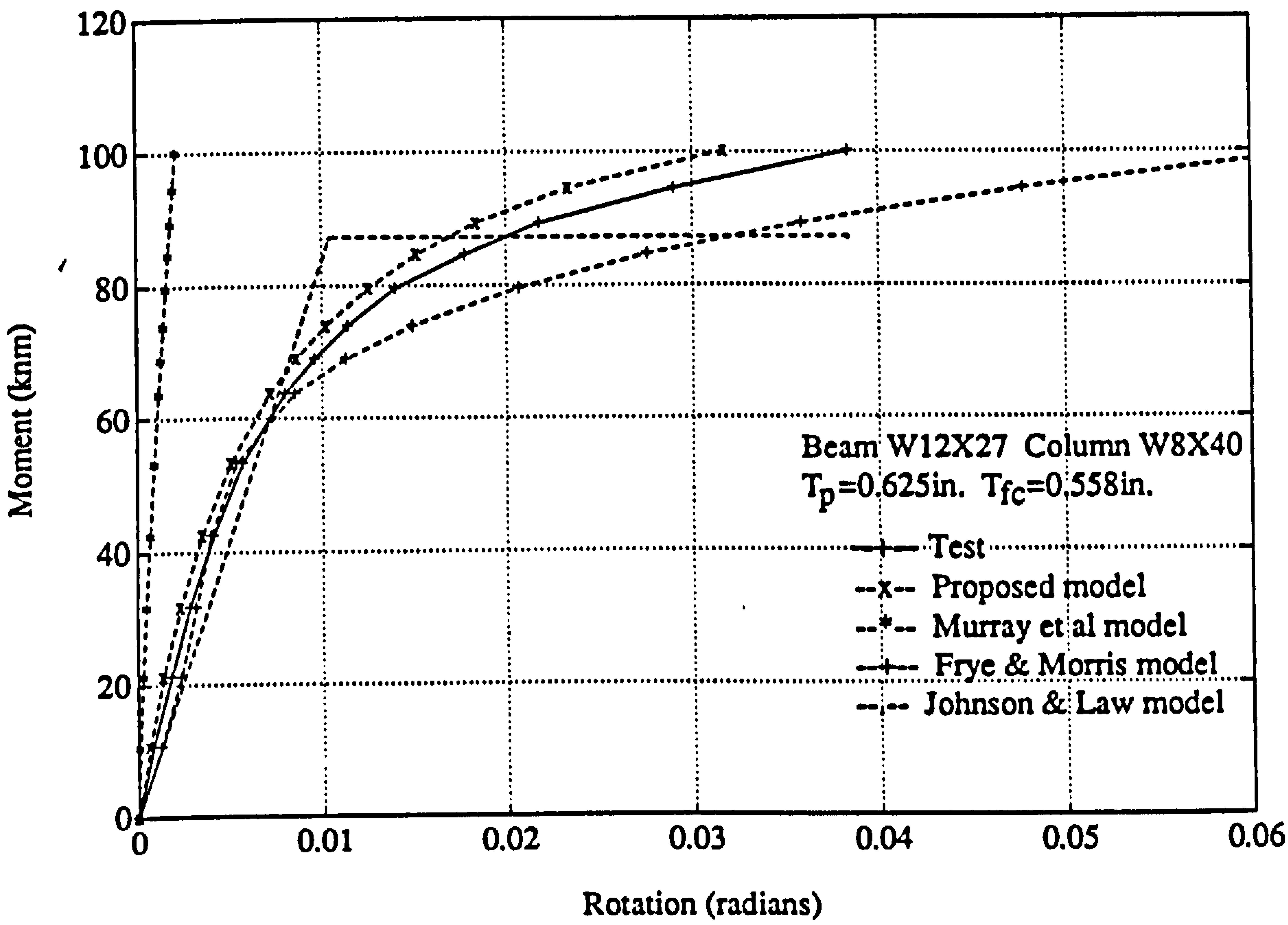


FIG.3-24 Comparison between analytical moment-rotation relationships and Ostrander's test 17 (unstiffened column).

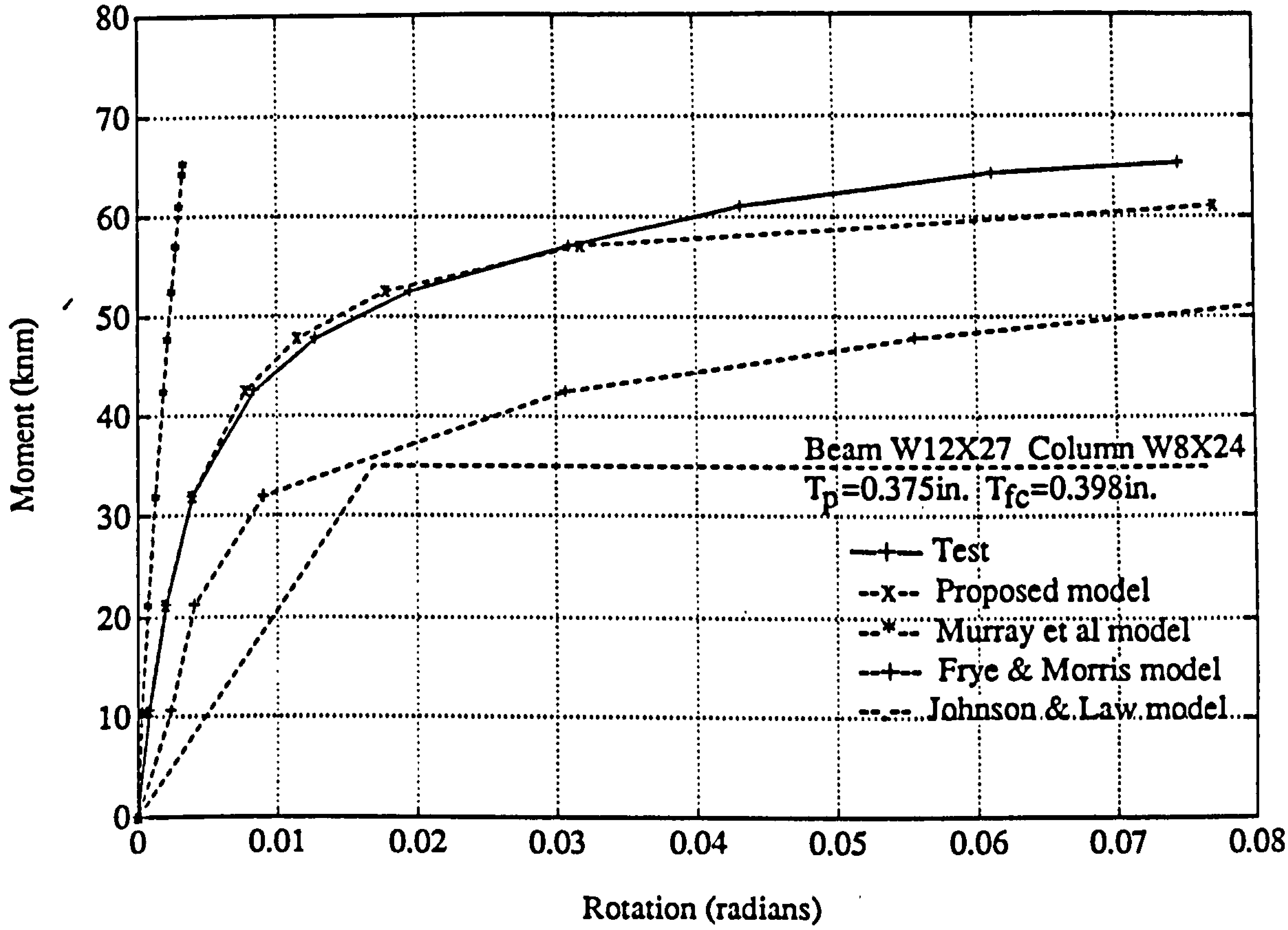


FIG.3-25 Comparison between analytical moment-rotation relationships and Ostrander's test 18 (unstiffened column).

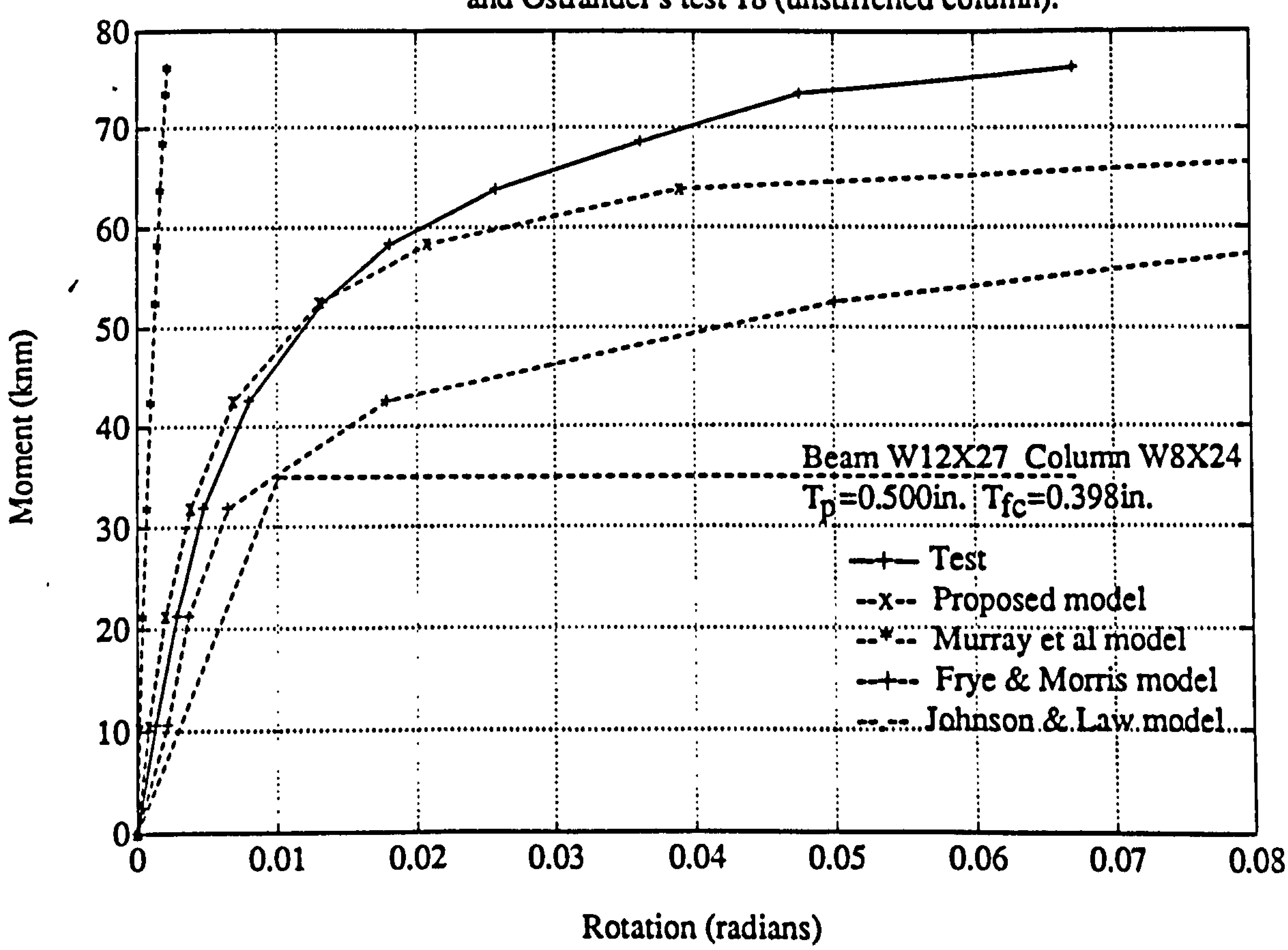


FIG.3-26 Comparison between analytical moment-rotation relationships and Ostrander's test 19 (unstiffened column).

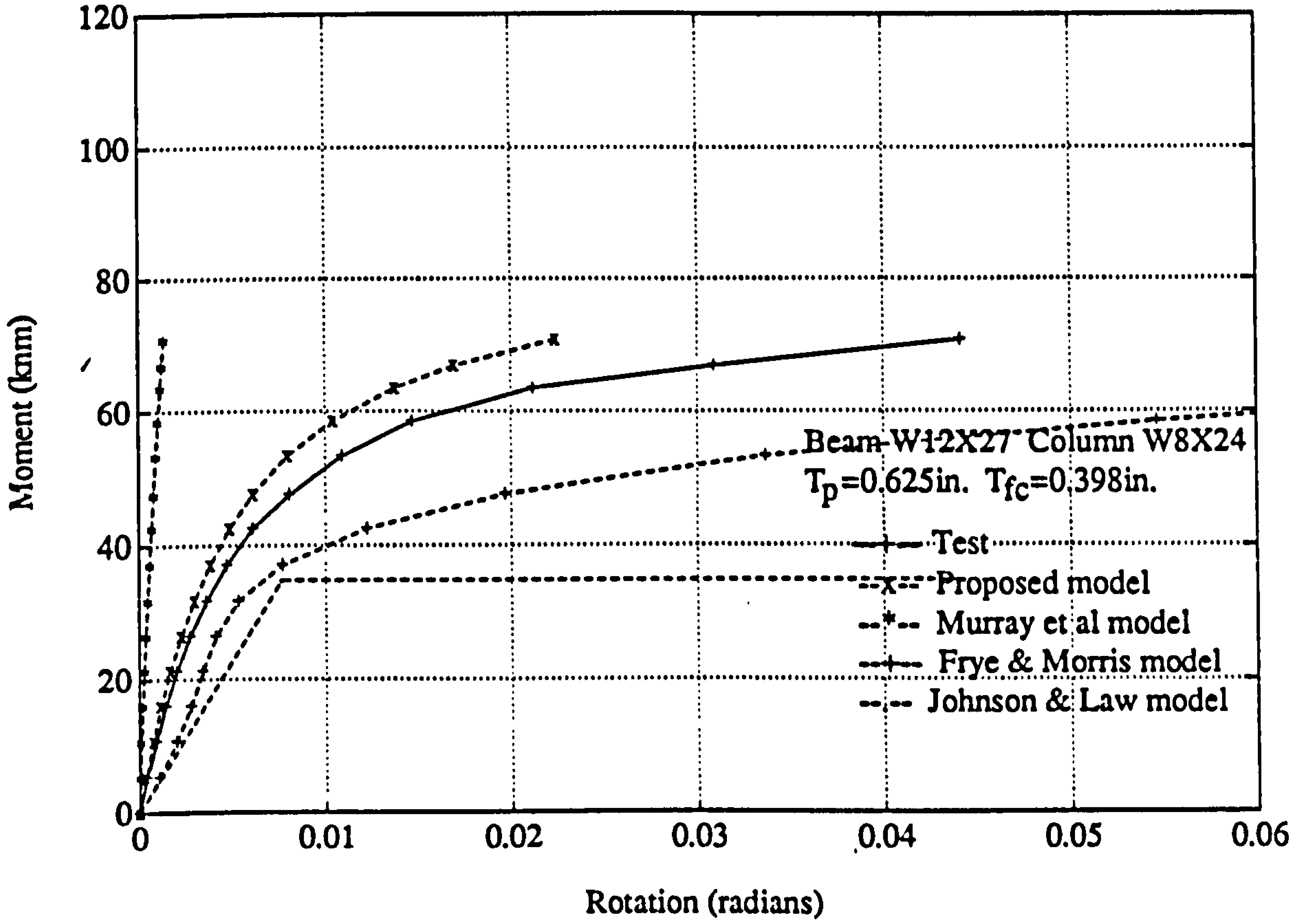


FIG.3-27 Comparison between analytical moment-rotation relationships and Ostrander's test 23 (unstiffened column).

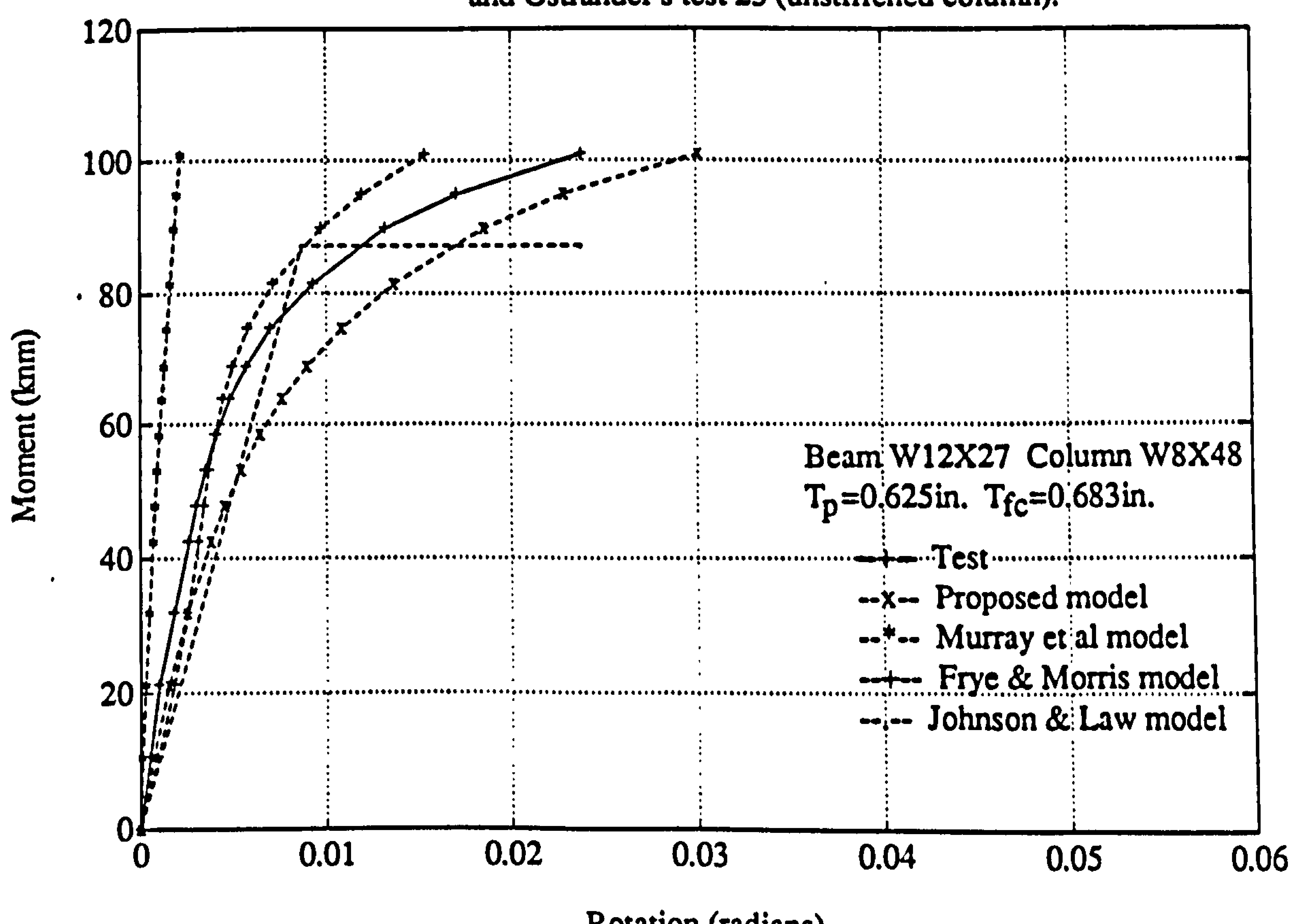


FIG.3-28 Comparison between analytical moment-rotation relationships and Zoetemeijer & Kolstein test 12(unstiffened column).

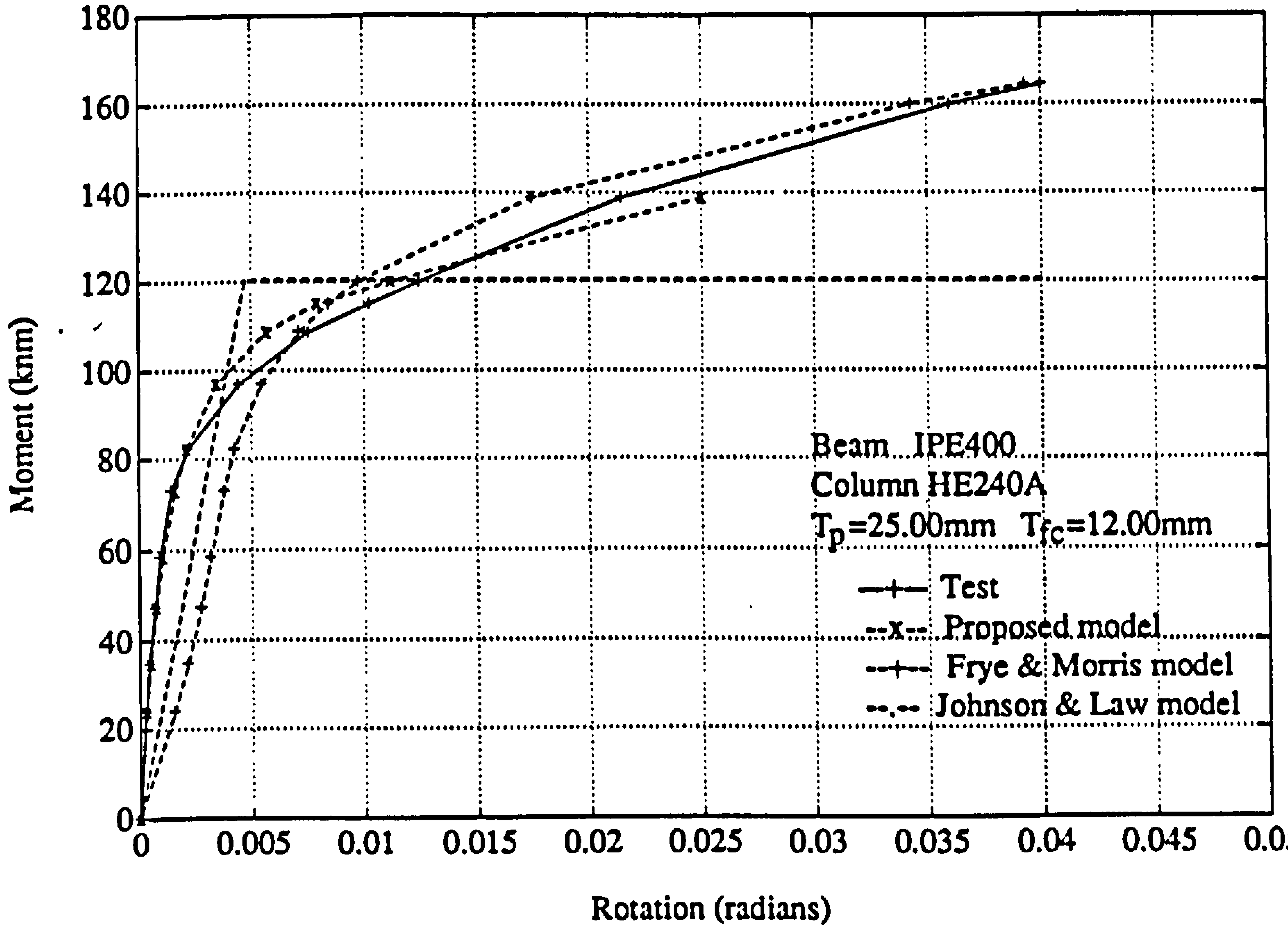


FIG.3-29 Comparison between analytical moment-rotation relationships and Zoetemeijer & Kolstein test 18(unstiffened column).

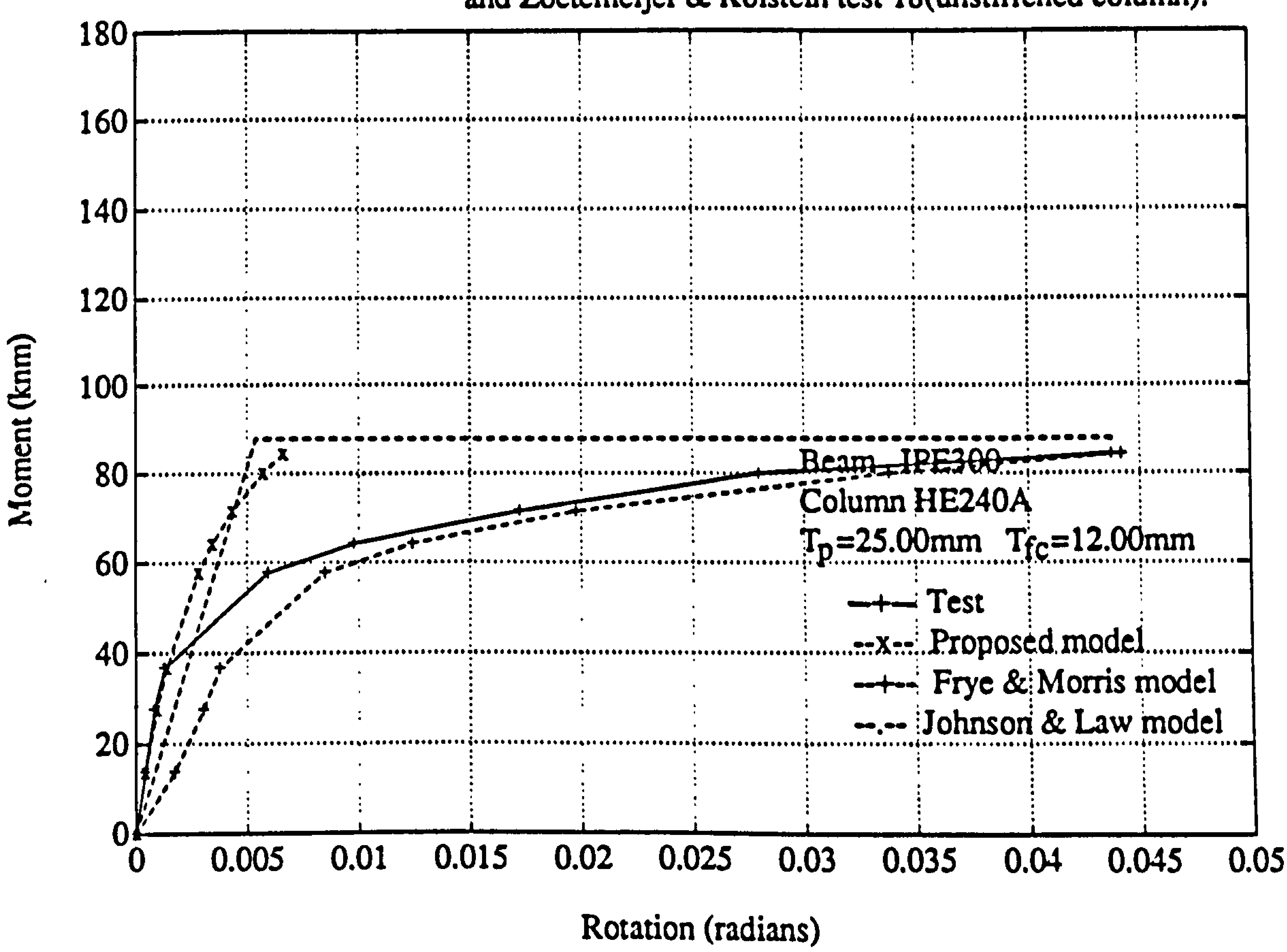


FIG.3-30 Comparison between analytical moment-rotation relationships and Davison's test JT12 (unstiffened column).

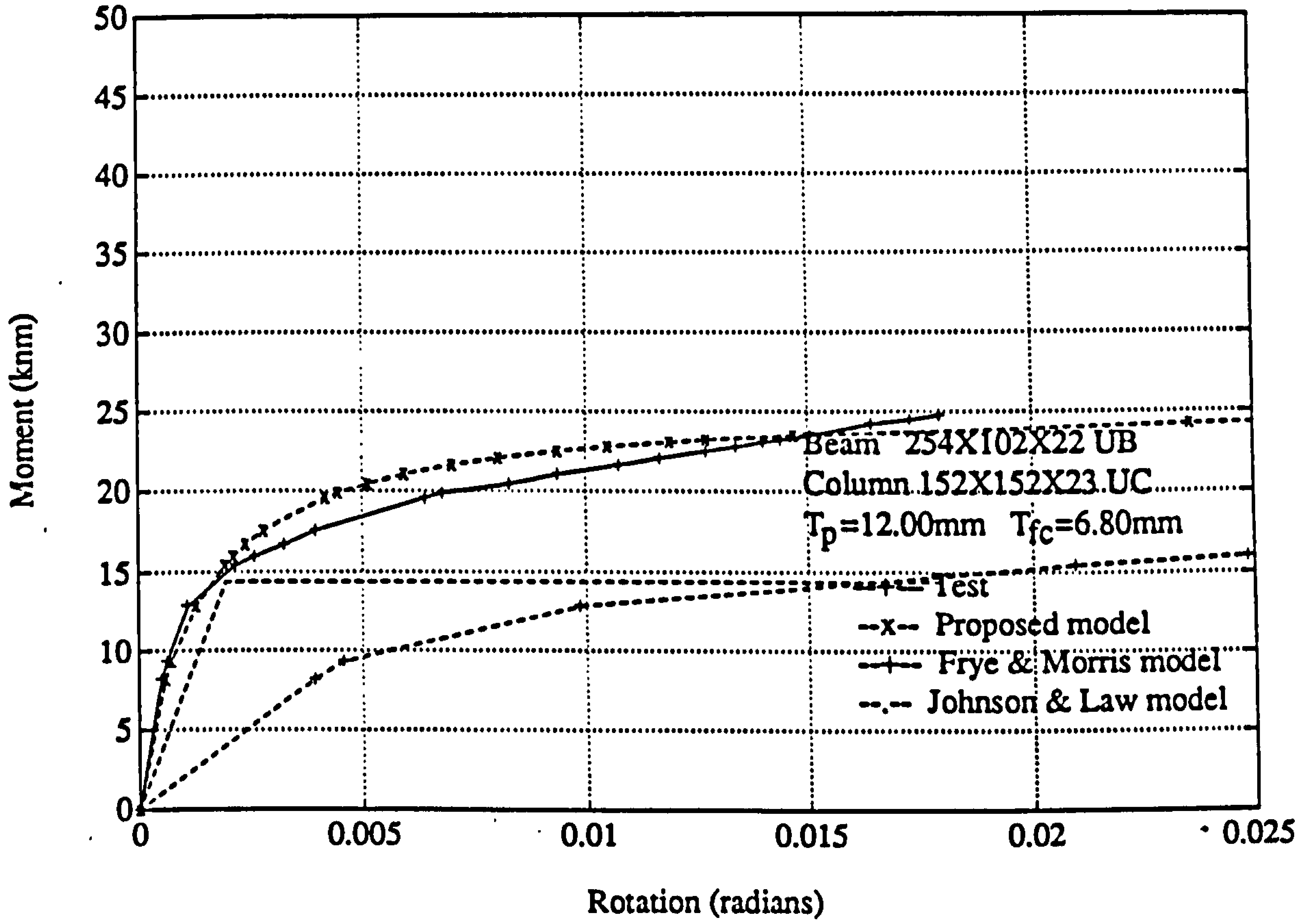


FIG.3-31a Comparison between Frye & Morris's model and Ostrander's tests 2,5,&10 (stiffened column).

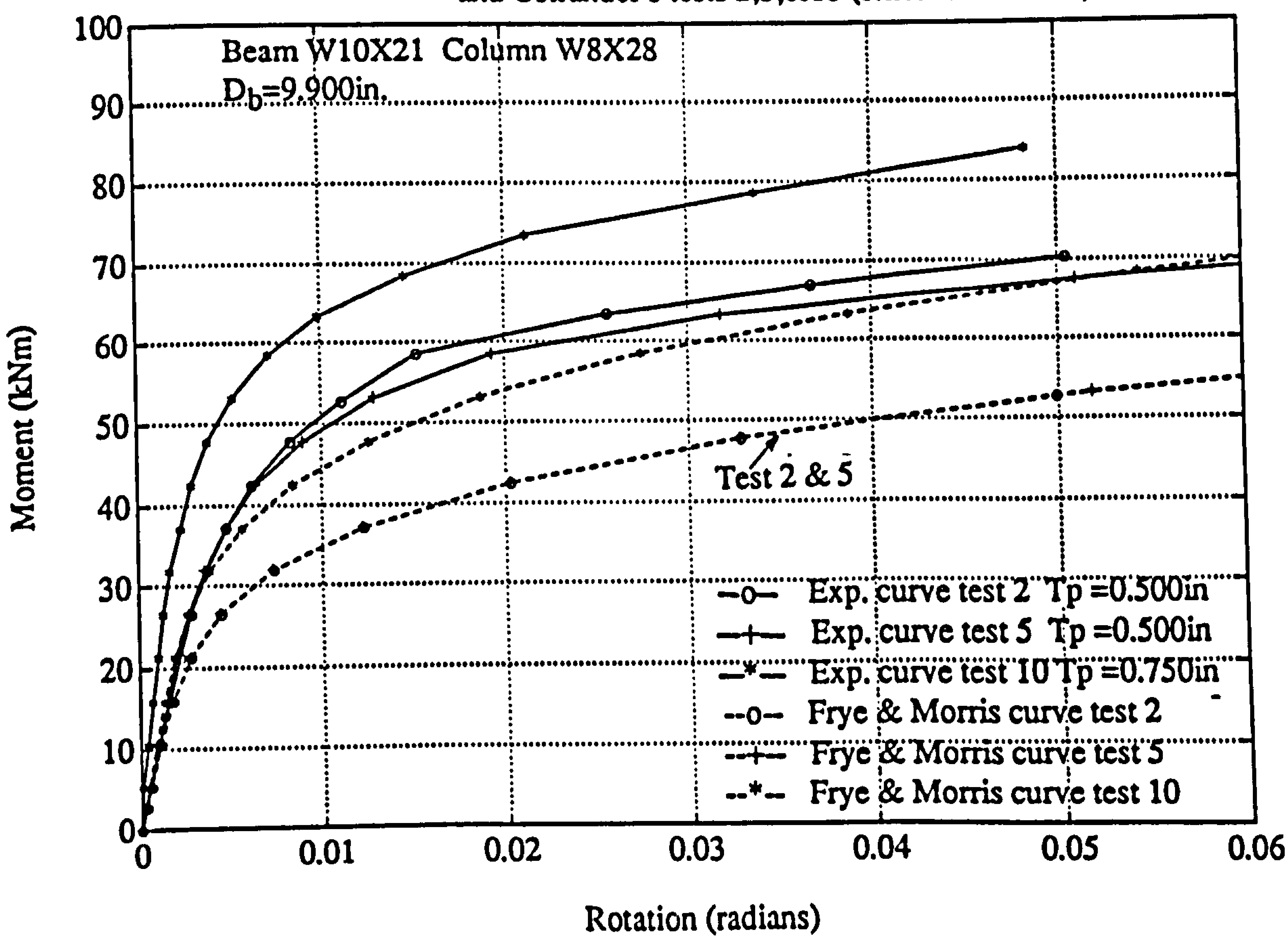


FIG.3-31b Comparison between Frye & Morris's model and Ostrander's tests 6, 7 & 8 (stiffened column).

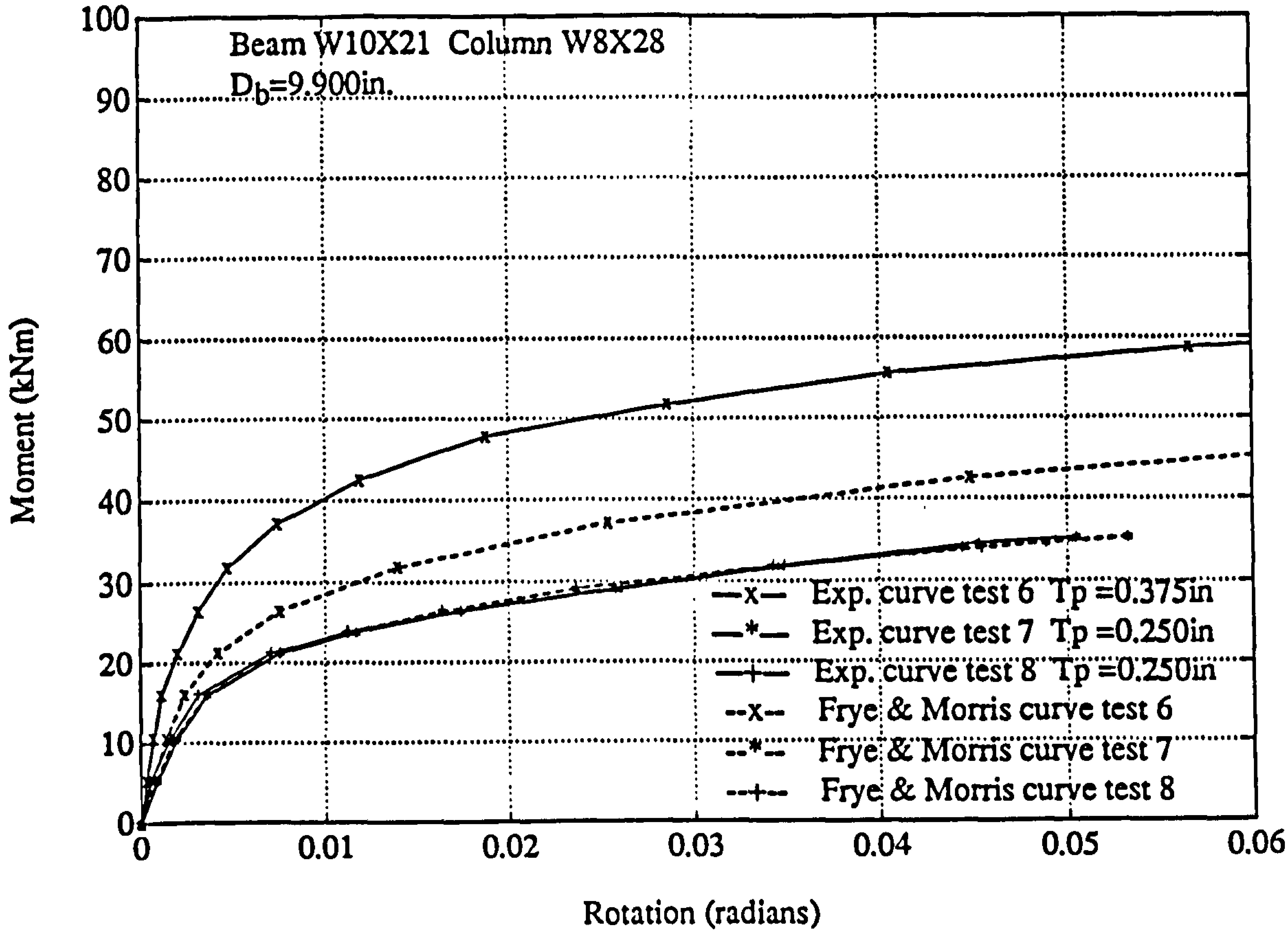


FIG.3-32 Comparison between Frye & Morris's model and Ostrander's tests 14,15 & 16 (stiffened column).

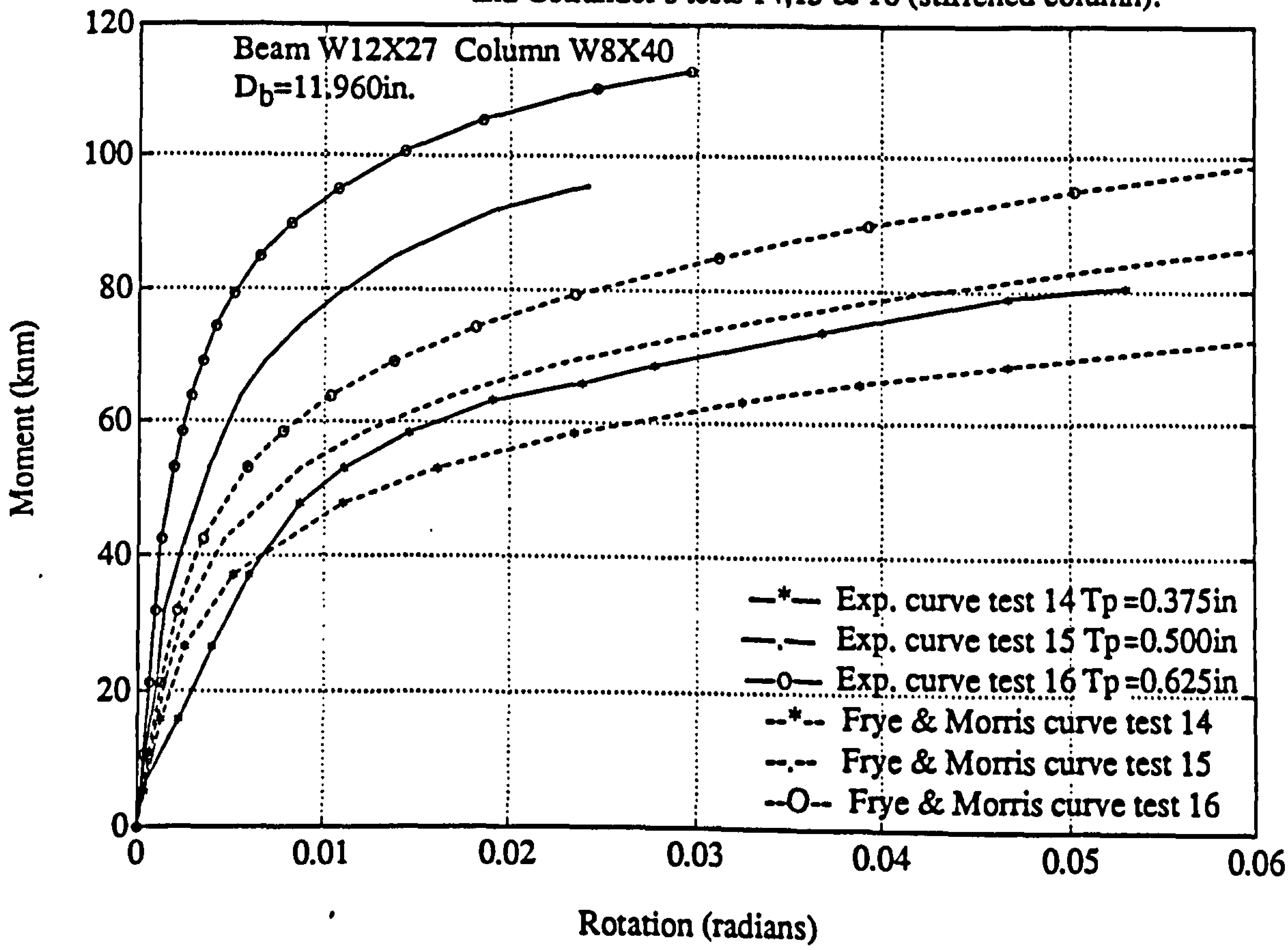


FIG.3-33 Comparison between Frye & Morris's model and Ostrander's tests 20,21 & 22 (stiffened column).

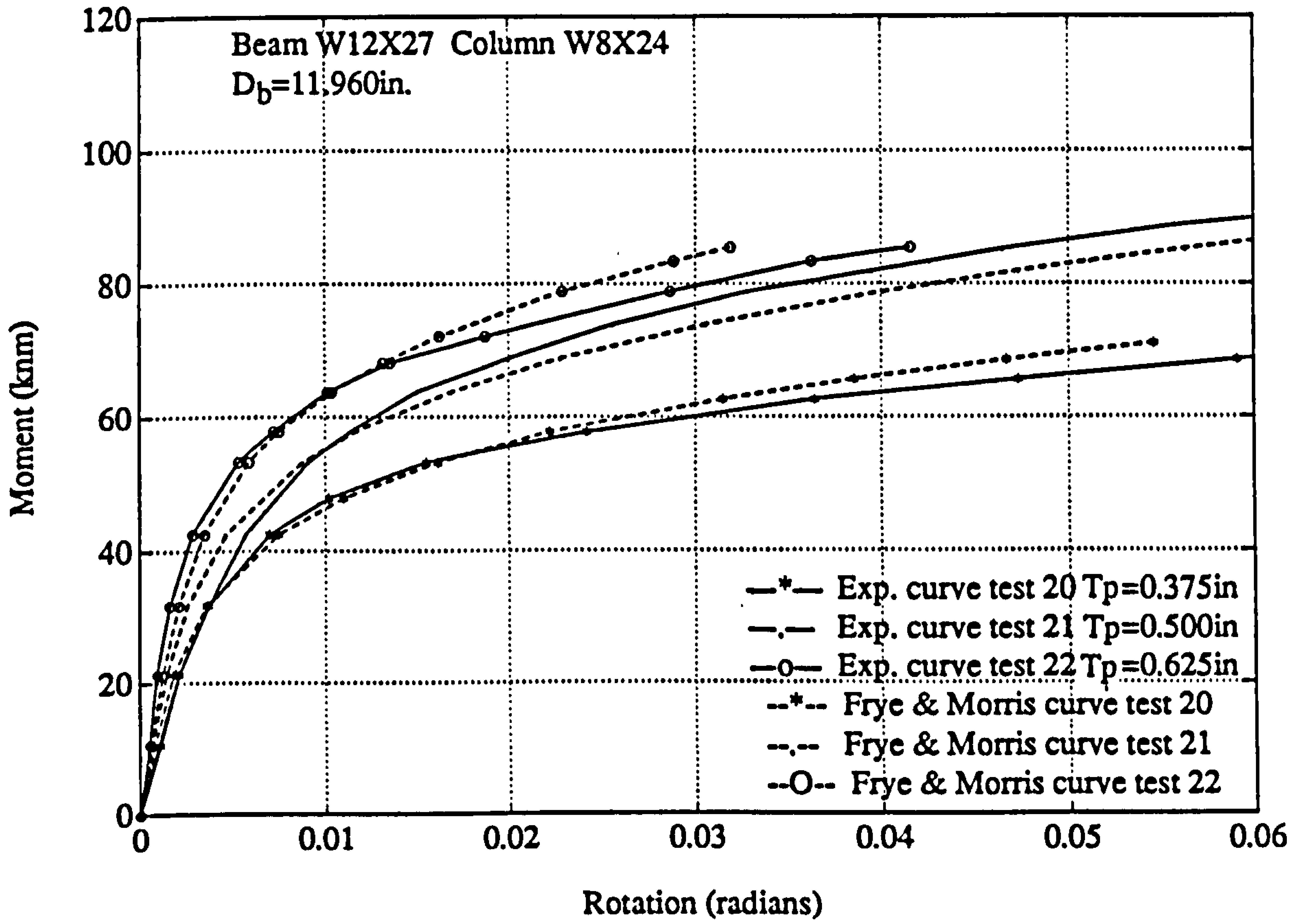


FIG.3-34 Comparison between Frye & Morris's model and Ostrander's test 24 (stiffened column).

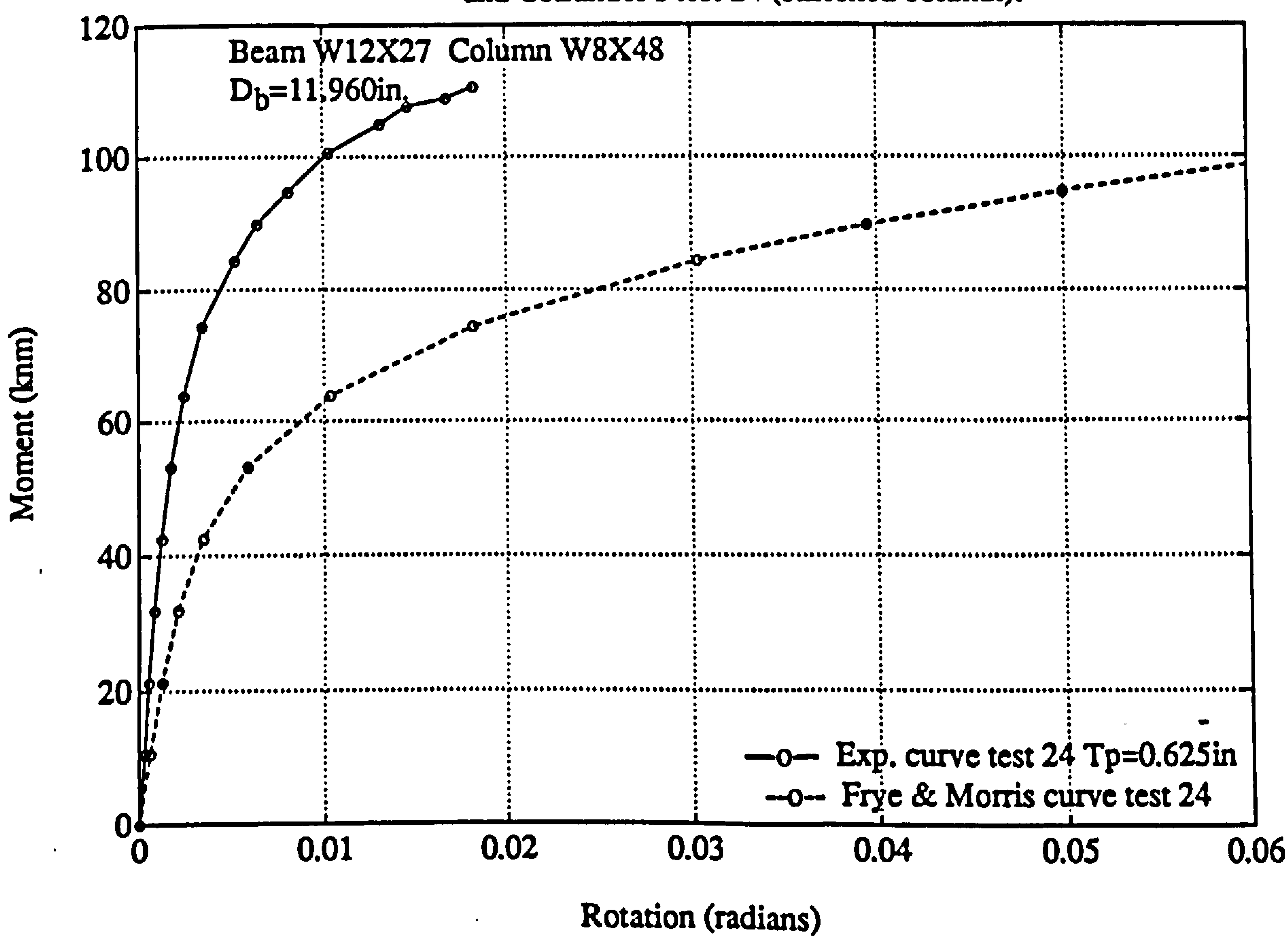


FIG.3-35 Comparison between Frye & Morris's model and Prescott's tests 19,20 & 21 (stiffened column).

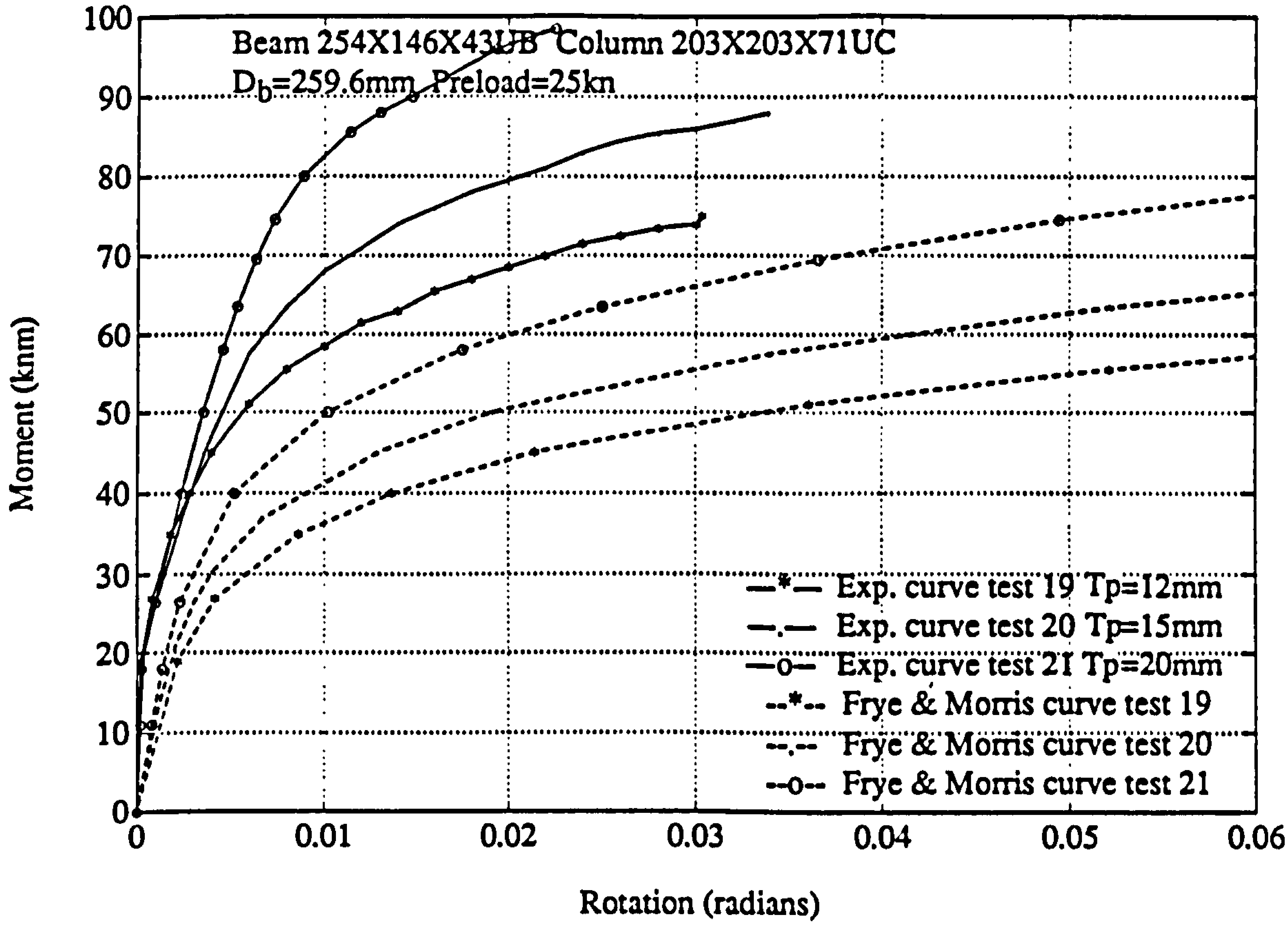
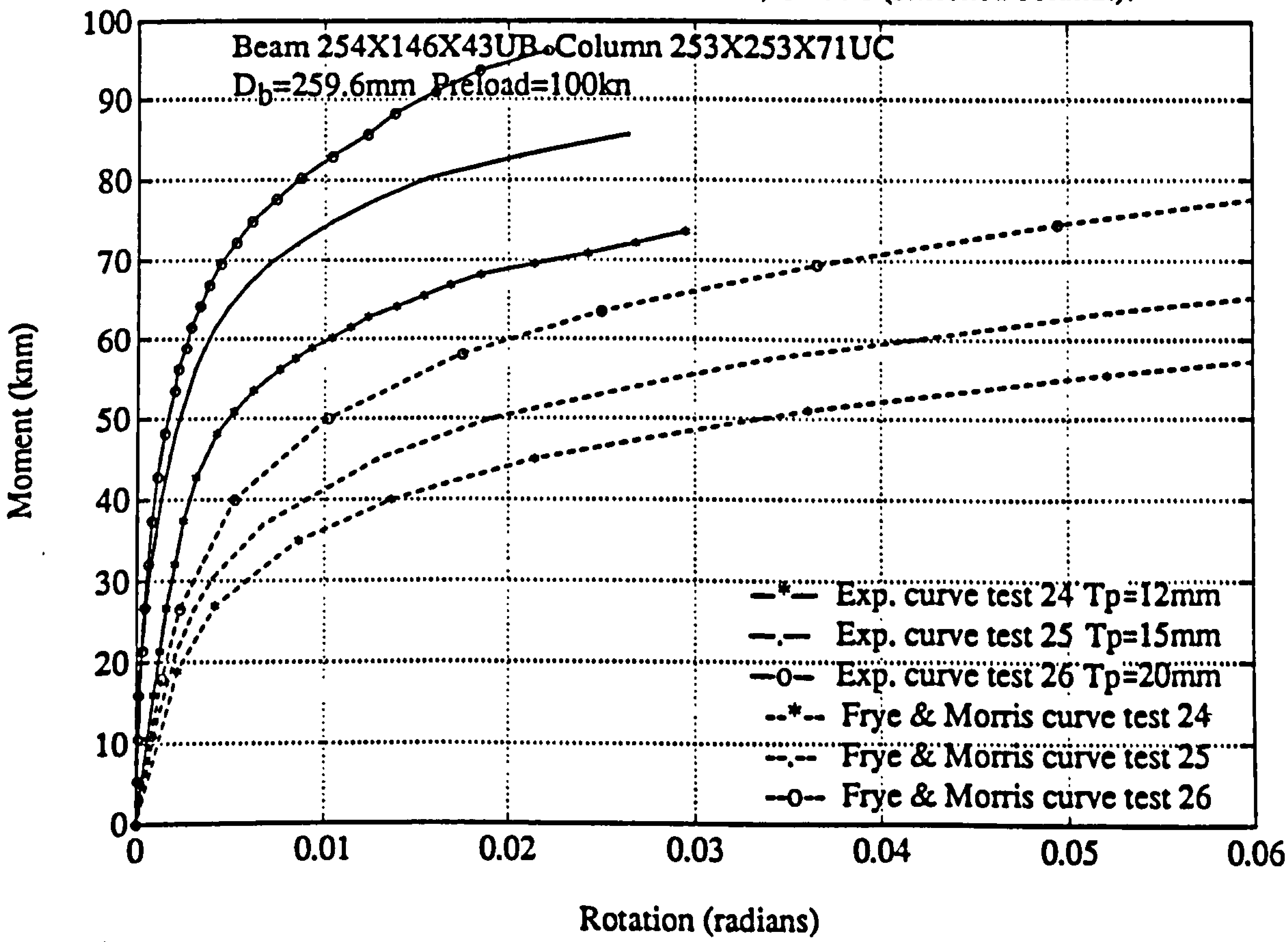


FIG.3-36 Comparison between Frye & Morris's model and Prescott's tests 24,25 & 26 (stiffened column).



CHAPTER IV

EXTENDED END-PLATE CONNECTIONS

4.1 Introduction

Bolted extended end-plate joints have been a common form of connecting beams to columns in rigid-frame construction, as such joints can be designed to satisfy the necessary criteria of strength, stiffness and rotation capacity, yet still remain economical and easy to construct. In such connections, the end-plate can be extended either on the tension side only as shown in Figure 4.1a or on both sides as in Figure 4.1b. Due to its symmetry the latter alternative is preferred when the frame is subjected to reversal of loadings, such as earthquake or wind loading. The main disadvantages of the extended end-plate in comparison to the flush end-plate connection is that part of the connection now intrudes above and possibly below the beam's section. Both forms of the extended end-plate can be either with or without column web stiffeners; these are used to prevent the flexural deformation of the column flange. Different degrees of moment-transfer are expected from these connections. For an unstiffened connection, typically the flush end-plate connection transmits approximately 30% and the extended end-plate connection transmits about 60% of the beam's yield moment.

Because of the extra row of tension bolts outside the beam flange, the effective lever arm of the resultant tensile force will be increased in comparison to the flush end-plate connections and consequently greater connection stiffness and strength can be achieved. As the effective lever arm to the bolts increases as the end-plate extends and as the rotation normally occurs about the beam compression flange, this leads to smaller bolt sizes with a decrease in the corresponding tensile and compressive forces being applied to the column. Thinner end-plates are normally associated with this arrangement, resulting in the extended end-plate system being usually preferred in practice.

This type of connection has received the widest study because of its popularity in recent years and is common in many modern steel structures.

The notation for the main parameters of extended end-plate connections as shown in Figure 4.1 are identified as follows:

L_p = Plate depth,

B_p = Plate width,

T_p = Plate thickness,

T_{wb} = Beam web thickness,

D_b = Beam depth,

T_{fc} = Column flange thickness,

T_s = Column web stiffener thickness,

G = Distance between two lines of fasteners, referred as gauge distance,

C_t = Distance from the tension side edge of plate to the centre of the outer tension side fastener holes,

P_t = Distance between two rows of tension side fasteners, and

L_i = Distance from the centre of the lower tension side fastener, holes to the centre of the upper compression-side fastener holes.

For extended end-plate connections on tension side only (Figure 4.1a):

P_c = Distance from the inner row of fasteners on the compression side to the centre of compression-side flange of the beam, and

C_c = Distance from the centre of the compression-side flange of the beam to the compression-side edge of the plate.

For extended end-plate connections on both the tension and the compression sides (Figure 4.1b):

P_c = Distance between two rows of compression-side fasteners,

C_c = Distance from the outer row of fasteners on the compression side to the compression-side edge of plate,

P_{it} = Distance from the inner row of fasteners on the tension side to the upper row of the central fasteners,

P_i = Distance between the two inner rows of the central fasteners,

P_{ic} = Distance from the inner row of fasteners on the compression side to the lower row of the central fasteners, and

L_b = Distance between the extreme bolt's centres.

For extended end-plate connections with only two bolt rows on the tension side and one bolt row on the compression side, P_i represents the distance between the inner bolt row in tension and the row of bolts in compression. In this case, the parameters P_{it} and P_{ic} are equal to zero.

4.2 General behaviour and design recommendations

The moment-rotation curve for this form of connection is quite similar to that discussed previously for the flush end-plate, see section 3.2, but with greater initial stiffness and moment capacity. Figures 4.2a and 4.2b clearly show the difference in behaviour between extended and flush end-plate connection for the tests of reference [4.1] where all other parameters were similar for the two specimens. The extended end-plate is seen to be far stiffer at all load levels and possesses an ultimate moment of about 1.6 times greater than the flush end-plate, irrespective of bolt preload. Initially, the moment-rotation curve is almost linear, followed by a large knee, during which considerable softening of the connection occurs, leading to a final phase of approximately 5 to 10 per cent of its initial stiffness. The shape and the extension of the curve in the latter range depends upon the connection component which principally is affected by yielding.

The behaviour of end-plate connections with stiffened column flanges and web differ greatly from an end-plate connection to an unstiffened column. The stiffeners are specially designed to strengthen the column web in compression zone and to minimise the possibility of premature failure by column web buckling. However, their presence in the tension zone tends to restrain the column flanges and increase the moment-capacity of the joint.

Similarly to the flush end-plate connections, the connection rotation arises from a complex interaction of end-plate bending, column flange deformation (due to bending and shear), direct force in the column web and bolt stretching. The source of large deformation is dependent upon the relative thickness of the end-plate to the column flange thickness. A thicker end-plate will reduce the component of rotation attributable to plate flexure but bolt length, and consequently bolt extension would be increased. Unless there is a significant contribution from the column, only limited rotation is provided, by the bolt extension. The limited ductility, typical of high strength bolts, tends to affect negatively the rotation capacity of the overall connection. Based on experimental investigation results of extended end-plate connections, a thick end-plate may seriously restrict the rotation capacity of the connection, i.e. its ability to deform plastically. Such behaviour must be avoided at the ultimate limit state due to sudden failure of the joint. The predominant mode of failure for such type of connections is by bolt fracture in tension.

However, for thin end-plate connections, the majority of the rotation comes from the bending of the end-plate. Experimental evidence has shown that the maximum bending of the end-plate occurs in the tensile region between the top two bolt rows. Zandonini and Zanon [4.2] reported that the additional rotation due to end-plate bending may contribute up to 95% of the total connection rotation. The predominant mode of failure for such type of connections is by end-plate failure and/or failure of column flange in flexure. The ultimate moment of the connection will be less than that based on bolt fracture but much higher rotation can be achieved. For a stiffened column, the contribution of the column flange to connection rotation is negligible.

The moment capacity of a bolted beam-to-column connection will depend on the strength of its individual elements. The moment acting in such connections may be replaced by a couple whose forces act at beam flange level. The lowest value of the force F_p obtained by considering the criteria associated with the failure modes will determine M_p. Most of the analysis and design procedure examine the behaviour of the connection by identifying different failure mechanisms, determining the appropriate mode of failure and analysing the mechanism for the present forces. Various investigations conducted in the past which will be reviewed in section 4.3.1, have shown the following mode of failures:

a) **In the tension region:**

i) *Failure of column flanges in flexure in the tension region*:

In the tension region of an extended end-plate connection the flexibility of the column flange, as well as that of the end-plate, plays an integral part in the ultimate failure of the connection. The tensile beam flange force is transmitted through the bolts to the column flange, and induces flexure of the column flange and tension in the web. Since the load is applied through both the upper and lower tension bolt rows, the web tensile stress is well dispersed and hence low, causing column flange flexure to be the mode of failure [4.3]. For stiffened connections such mode of failure does not occur because of the restraint provided by the column stiffeners, positioned between the top bolt rows situated on either side of the tension flange.

Packer and Morris [4.4] studied the behaviour of column flanges which were decidely less stiff than the end-plate. They examined two modes of failure for unstiffened column flanges. The first mode occurs when both column flanges are yielding adjacent to the web and bolt fracture occurs. The second mode of failure occurs when a mechanism with double curvature forms. The lower value of the two modes of failure will indicate by which mode the column flange will actually fail.

ii) *Column web behind the beam tension flange*:

Usually the design of the column web in the tension zone is not critical if the column web is designed to resist the tension force. In other words, the column web must resist the tension force in the beam tension flange without fracture or excessive yield. If not, then stiffeners are required in order to reduce the stress concentration in the web transmitted through the bolts via the end-plate.

iii) *Bolt failure*:

This mode of failure is generally associated with relatively thick end-plates. The bolt tension is a combination of direct force and induced prying force. The direct force results from the applied moment and the prying force from the flexural distortion of the end-plate and column flange. In other words these additional forces induced in the tension bolts are the results of the bearing of the top edge of the end-plate against the column flange. Therefore, the total force in the bolts is the sum of the applied force F_t and the induced prying force. The distribution of both sets of force to the individual bolt rows relates to the stiffness of the surrounding end-plate.

The prying force concept which forms the basis for the existing T-stub and end-plate design formulae was originally suggested by Shultz [4.5]. Douty and McGuire [4.6] refined Schultz's suggestions and through analytical and experimental investigations arrived at semi-empirical formulae for the prediction of the prying force in terms of the dimensions of the T-stub. Based on additional research, Nair et al [4.7] developed an alternate formulae and provided the prying force equation.

Based on Packer's investigation [4.3], Packer and Morris [4.4] gave design recommendations for stiffened and unstiffened column flanges bolted to extended end-plate connections. They have also verified experimentally the empirical value of the total bolt force suggested by Surtees and Mann [4.8] who assumed a prying force of 33%.

(iv) *End-plate weld failure*:

The welds that connect the end-plate to the beam member are critical. Mann and Morris [4.9] recommended the use of fillet welds rather than butt welding not only to eliminate the cost of edge preparation but to reduce the risks associated with lameller tearing. It is recommended that the throat thickness must not be less than one half the beam flange thickness for flange welds and one half the beam web thickness for web welds, provided that the heavier flange weld is continued down the web for at least 50 mm in the tension region [4.9].

v) *End-plate failure*:

A considerable amount of work has been done on the determination of the strength of the end-plate connection in references [4.4, 4.6, 4.8, 4.9, 4.10, 4.11, 4.12, 4.13, 4.14, 4.15, 4.16]. The critical region of the end-plate is identified as being the tension zone where the beam tension flange meets the end-plate. The prediction of the end-plate's resistance may be based on a complete end-plate yield line mechanism as shown in Figure 4.2c or idealised tee-stub mechanism as shown in Figure 4.2d. In the latter mechanism, plastic hinges form at both the bolt line and the web junction of the tee-stub flange plate.

As a result of these modes of failures in the tension region, the critical design parameters are the column web thickness, the column flange thickness, the gauge and pitch of the bolts and the thickness of the end-plate.

b) **In the compression region:**

i) *Buckling of the column web*:

The column web must be capable of resisting large localised tensile and compressive forces. In the compression zone checks must be made for web crippling and for web buckling. Web crippling is characterised by yielding of the web in compression. Research on the study of column web buckling strength in beam-to-column connections was carried out by Chen and Newlin [4.17] and a web buckling formula was suggested by Chen and Oppenhein [4.18].

ii) *Web crippling*:

Graham et al [4.19] studied the web crippling criterion for welded beam-to-column connections which later was modified by Witteveen et al [4.20] for bolted end-plate connections. The strength and rotation capacity of the connection are influenced by web buckling and crushing.

The critical design parameters for the compression region are the column web thickness, depth of the column, the distance from the outer face of the column flange to the web toe of fillet, and the thickness of the end plate [4.21].

c) *Shear yielding of the web in the shear region*

The couple acting on the tensile and compressive beam flanges can lead to very high shear stresses occurring in the web panel of the connection. This situation happens when beams framing into opposite sides of a column have a large difference in end-moment or for external columns where only one beam frames into a column. This leads to an out-of-balance moment acting on the column member which in turn creates a shearing action which has to be resisted by the column web. Therefore, the shear stress in the column web panel must be checked.

For beams framing into opposite sides of a column and having a similar end moment, the design criteria for the column web are the tearing and yielding in the tension region (described in section 4.2.a.ii)) and buckling or crushing in the compression zone (described in section 4.2.b.i).

The evaluation of connection capacity can be considered into four parts; beam end-plate, bolts, column and stiffening; although in practice the selected end-plate thickness and bolt size affect the strength of other parts of the system. However, for design purpose it is necessary to size and evaluate the components of the connection independently, and to ignore interactive effects. For a more comprehensive review of previous theoretical and experimental investigations and existing design formula for bolted moment connections the reader is referred to the work by Horne and Morris [4.16].

Because of the large number of variables present, comparison between results of different test series is difficult. However, the author felt necessary to conduct a parametric study to quantify the effect of varying connection geometry on the initial stiffness and complete moment-rotation behaviour of the connection.

4.3 Parametric analysis

The behaviour of beam to column connections have been investigated by several researchers and the extended end-plate joint has received much attention in recent years. This type of rigid connection became more favourable than joints formed with the tee stubs because of the reduced amount of material and labour involved. In total 139 test specimens from 20 different investigations have been identified. These data include extensions on one or both sides of the beam, although much of the data is for an extension on the tension side only. The collected data for this particular type of connection is listed in Table 4.1 and a complete description of each test specimen with its numerical form of moment-rotation data is presented in Appendix C.

4.3.1 Experimental data

Johnson et al [4.22] aimed at showing the adequacy of a variety of connections employing high strength bolts in tension and shear. Only on one test specimen was an extended end-plate connection used. The ½ in (12.7 mm) thick end-plate which extended beyond the tension flange was welded to 10 x 4½ x 25 beams 54 in. (137.6 mm) long and bolted to 8 x 8 x 45 column stub by four rows of two ¾ in (19 mm) A325 bolts. Column web stiffeners of 1/2 in (12.7 mm) thickness were provided opposite both beam flanges. The load was applied at the column stub of the cruciform arrangement. Johnson et al [4.22] confined their attention to measurement of moment-deflection relationships. The moment-rotation data was derived from moment-deflection curves by subtracting the vertical deflection due to a simple beam and dividing the result by half the clear span [4.23]. The investigators demonstrated that extended end-plate connections will transmit bending moment at least equal to the full plastic moment of the adjacent beam, without excessive deformation. The moment-rotation data of the tested connection is provided in numerical form in references [4.23, 4.24].

Sherbourne [4.10] conducted an experimental investigation of beam-to-column end-plate connections that focused on the ability of columns to develop the full plastic moment capacity of the connecting beams. The beams were 15 x 5 UB42 and the column stubs 8 x 8 UC35. A series of five tests were investigated, the first group of three tests using ¾ in (19 mm) diameter high tensile bolts in conjunction with ¾ in (19 mm) diameter black bolts. The second series of tests used 7/8 in (22 mm) diameter high tensile bolts. The end-plate extended on the tension side only with a thickness ranging from ¾ in (19 mm) to 1¼ in (32 mm). In four specimens column web stiffeners opposite both beam flanges were provided and the thickness varied from 5/16 in (8 mm) to 5/8 in (16 mm) thick. Only the load-deflection characteristics were recorded. This measured deflection corresponded to the vertical deflection of column stub. It was concluded that nominal amount of stiffening was required for columns with thin flanges. No recommendations were given as to the flange thickness below which stiffeners were required. He also concluded that the maximum restraint developed by this particular type of connection was approximately 70% of full rigidity and this restraint appeared to be independent of the thickness of the column stiffeners. The moment-rotation data for the five specimens are provided in references [4.23, 4.24].

In 1968 for his PhD thesis, Mann [4.25] attempted to determine the influence of beam to column connections on the strength and stiffness of plastically designed rigid steel frames. Six tests were performed on a one single sided beam-to-column connections, as opposed to balanced double beam connections investigated previously by other authors. The lower edge of the beam end-plate rested on a shear plate welded to the column flange to transmit the full vertical shear force from the beam to the column and therefore to eliminate the use of friction grip bolts to resist the vertical shear. Details of test specimens are given in Appendix C.

The absence of stiffeners to the compression zone of the column web in test specimen C1 resulted in premature plasticity in the column and beam and contributed to poor connection stiffness. As the connections were extremely stiff, some minimal stiffening to the column was required by reinforcing by half length column web stiffeners in the compression zone in tests C2 to C5. These tests differed mainly in end-plate material and thickness. The latter had a significant effect on the mode of failure and magnitude of failure moment.

The last specimen was provided with full length stiffeners in the compression and tensile region and a deep beam section was used. Based on end-plate failure as the collapse mode, Mann [4.25] developed an equation for the end-plate thickness. Such mode of failure is valid only if the column flange is relatively thick in relation to the end-plate and this was the case for test specimens carried out by the investigator.

An extensive experimental investigation was carried out by Bailey [4.26] to study the behaviour of extended end-plate connections bolted to the column flange with high strength friction grip bolts. In total, 13 pairs of extended end-plate beam to column connections were designed in accordance with the equations developed by Sherbourne [4.10] and grouped in three test series A, B and C. The specimens were tested under two types of loading; maximum moment using the maximum lever arm of 48 in (1219 mm) or maximum shear using the minimum lever arm of 18 in (457 mm).

In all specimens, the column section was kept constant, 8 in x 8 in x 48 lb (216 mm x 216 mm x 71 kg/m) while beam sections, end-plate dimensions and friction grip bolt size were varied. The three specimens of series A were fabricated from high yield steel (Grade 50B) while test specimens of series B and C were fabricated from mild steel (Grade 43A). Test specimens of series A and B were subjected to maximum moment conditions while the other five specimens of series C were subjected to maximum shear loading. No stiffeners were used in the three assemblies of series A while 5/16 in (8 mm) thickness column web stiffeners opposite both beam flanges were applied in test specimens of groups B and C to prevent premature failure owing to local instabilities in the column web.

Bailey concluded that there are no benefits to be gained in using HSFG bolts in the tension zone. High tensile bolts to normal spanner tightness would be satisfactory. He also suggested that loading conditions which produce average shear stresses only marginally reduce the plastic moment of resistance of the connection. The average rigidity of the connections in group A was found to be approximately 60% full rigidity with sufficient rotation capacity.

No attempt was made to measure accurately the rotation. The available moment-deflection curves in reference [4.26] were converted approximately to moment-rotation curves by Goverdhan [4.23] by assuming the lever arm of 48 in. (1219 mm) for the maximum moment tests and 18 in (457 mm) for the maximum shear tests, except for test C12 and C13 where lever arms of 24 in (610 mm) and 25 in (635 mm) respectively were used. He also assumed that the deflection was recorded at the tip of the beam and the beams were 67 in (1702 mm) long.

In order to establish if adequate rotation capacity occurred in a connection designed by the derived equations based on the results of tee stubs model tests, Zoetemeijer

[4.12] tested twenty three bolted beam-to-column connections. Only on four test specimens extended end-plate connections were used. All the connections were designed in such a way that collapse of the column flanges was the determining factor. The flanges of two test specimens were stiffened with bolted plates parallel to the flanges.

In addition of isolated beam-to-column connections, Zoetemeijer [4.12] also tested a full scale framework incorporating four joints and which were designed in accordance with the proposed equations. The results observed show that the joints had sufficient strength, stiffness and rotation capacity.

Moment-rotation curves for joints tested within the framework are available, however limited information is reported for the isolated tests in reference [4.12].

Packer [4.3] tested five beam-to-column joints in order to determine if the empirical formula developed for the tee-stub model, based on yield line theory to predict failure loads of such models, were applicable to extended end-plate connections. The tests incorporated 254 x 102 x 22 UB beams and 152 x 152 UC columns of serial weights 23, 30 and 37 Kg/m. A 15 mm end-plate thickness with 16 mm diameter HSFG bolts were used throughout the test programme. Four specimens were unstiffened around the tension zone while the column flange thickness varied. The column flanges of one connection were transversely stiffened with full depth stiffeners opposite both beam flanges while the remaining test represented a balanced beam to column joint in a multi-storey building. A constant axial load of 150 kN was applied to the column, and the beams were jacked to failure, in one test specimen. The effect of axial load in the column upon the column flange collapse load can be compared to a similar test specimen whose column was unloaded. Column yielding was found to be dependent upon the axial load carried by the column. Based on these results design recommendations for stiffened and unstiffened column flanges were reported in references [4.3, 4.4].

In 1978 for his PhD thesis, Ioannides [4.27] studied experimentally and theoretically the behaviour of the column flange in beam to column connections. The test specimen comprises two beams and a column to form a cruciform arrangement. The experimental test programme consisted of six tests, three with thin end plates and three with end-plates of medium thickness. The columns were unstiffened and the end-plate extended beyond the tension and compression flange of the beam. In all the tests the plates were bolted to the column with eight bolts. An axial load of approximately one third the nominal yield, axial load was applied to the top of the column. This was held constant throughout the test and loads were then applied to each of the beams at a lever arm of 4ft (1219 mm) to produce equal and opposite moments on the connections.

In addition a linear elastic analytical model was developed based on finite element analysis. The model includes the interaction between the end-plate and the column flange and models both the tension and compression region.

Three full scale extended end-plate, beam-to-column connections were tested at the University of Vanderbilt by Dews [4.28] in 1979. The columns were unstiffened and the end-plate extended beyond the tension and compression beam flanges. Each specimen had four bolts placed around the tension flange and four bolts placed around the compression flange.

Moment-rotation data is provided in references [4.23, 4.24] for the three test specimens.

In 1980, Grundy et al [4.11] reported the results of two cruciform extended end-plate, beam-to-column connections. Two end-plate thicknesses, 1 in (25 mm) and 1¼ in (32 mm) were used. The end-plate extended beyond the beam tension flange only, welded to 610 UB 113 beam sections and bolted to 310 UC 240 column section. A single row of four high strength bolts placed close to the compression flange. The moment versus deflection curves were presented. This data was converted to moment-rotation data in references [4.23, 4.24].

Four pairs of end-plate beam-to-column connections were tested by Johnson and Walpole [4.29] at the University of Canterbury, New Zealand, in 1981. The end-plate extended beyond the tension and compression beam flanges. The column web and flange were stiffened by doubler plates of 16 mm thickness. Only the load-deflection curves were reported. The envelope curves for the various cycles were converted to moment-rotation data in references [4.23, 4.24].

A series of experimental tests were carried out by Tarpy and Carbinal [4.21] to determine the effect of the end-plate forces on the column flanges and web. In total 16 extended end-plate, beam-to-column connections were tested. A series of beams, column sections and end-plate thicknesses were chosen. Only on two test specimens were column web stiffeners used. The end-plate thicknesses ranged from about 1.5 to 4 times the column flange thickness and they extended beyond the tension and compression beam flanges. An initial axial load of approximately one-third the column yield was applied to the column in most of these tests.

Limited information is available for these tests in reference [4.21], although it is believed that more detailed reports do exist in reference [4.30] which was unobtainable. The four M-Φ curves provided exhibit non linear behaviour throughout, with their initial stiffness being highly dependent upon the end-plate thickness. These tests are not comparable with many of the others in Table 4.1 as the end-plate extended above and below the beam. For all tests performed the failure mode was either by excessive rotations, connection yielding or web buckling. For tests with horizontal stiffners, failure in the column flange or web did not occur.

In addition to the experimental work, Tarpy and Cardinal [4.21] provided an expression for the moment-rotation characteristics of unstiffened extended end-plate connections. The finite element method used was based on a linear elastic model and equations predicting the behaviour of these joints were developed. The developed analytical moment-rotation relationships will be given in section 4.4.3.

In 1981 for his PhD thesis, Graham [4.31] exhaustively studied, both analytically and experimentally the unstiffened extended welded end-plate connected to the column flange with HSFG bolts. These internal beam-to-column connections were subjected to static loading conditions. The effect of varying the span, i.e. moment-to-shear ratio, and thickness of the end-plate and column flange for an unstiffened column section on rotation, slip, local deformation and prying forces in the bolts were shown by testing twenty one beam to column connections. The cruciform arrangement comprised of 356 x 171 x 45 UB and a column section consisted of a mild steel plate welded together to form a section of overall dimensions 200 x 200. The column had a constant web plate thickness of 12 mm to avoid premature failure due to its instability. The column flange thickness was varied in the tests from 11.6 to 17.5 mm. The end-plate extended on tension side above the beam flange only. Each joint had six bolts, two placed close to the compression flange while the remaining four were symmetrically placed around the tension flange and were tightened to an initial shank tension of 75 kN. His main findings from the experimental investigation were as follows:

i) The connections behaved in a rigid manner up to the elastic limit of either the end-plate or column flange.

ii) Rotation occurred about the beam compression flange.

iii) Gradual slip into bearing had negligible effect on the moment-rotation characteristics of the joints.

iv) Bolt failure loads were independent of the column flange thickness. However, the rotation capacity of the connections was dependent on the column flange thickness.

v) The yield strength of the plates had not only an effect on the range of elastic behaviour of the joints but also on their rotation capacity.

vi) For connections whose bolt diameter was larger than the thickness of both the end-plate and column flange, considerable rotation capacity was achieved before failure occurred.

Only 12 test specimen results are reported in reference [4.32]. Full details and moment-rotation data for the twenty one bolted beam-to-column connections are presented in reference [4.31] and are reproduced in Appendix C.

In 1981, Zoetemeijer [4.33] conducted an experimental test programme on bolted beam-to-column connections with haunched beams at Delft University. The testing was done in a cruciform mode with a column section of HE300A and an IPE400 for beam section. In total 10 specimens were tested to investigate the influence on M-Φ behaviour of haunched beam on compression side of beam and the effect of column stiffening for haunched connections.

Zoetemeijer [4.34] subsequently carried out five further tests on such connections. The cantilever arrangement comprised of HE300A column section and a beam section of IPE400. In two specimens web doubler plate on the column were used.

The main conclusions to be drawn from these studied are:

i) Moderate changes in end-plate thickness for end-plates with haunches on the compression side have negligible effects on the M-Φ curve until it becomes noticeably nonlinear at moments in excess of 60 per cent of connection capacity [4.34].

ii) Initial connection stiffness may be increased by using longer haunches and/or doubler plates which stiffen the column web over the full depth of the end-plate. Doubler plates in the compression region of the column only have little effect on M-Φ behaviour [4.33, 4.34].

As the extended end-plate connections are classified into two types in this thesis, namely extended on the tension side only or on both the tension and compression sides as shown in Figure 4.1, these specimens tested by Zoetemeijer [4.33, 4.34] cannot be classified into these categories of connections. This is because the end-plate extended only on the compression side of the beam and therefore M-Φ curves presented are not included in this collection.

Four extended end-plate connections were tested by Moore and Sims [4.35] at BRE, in 1983. Flange backing plates were used in two specimens as a cheaper alternative to the traditional web stiffener as suggested by Zoetemeijer [4.12]. A 254 x 102 UB22 was selected for the beam sections, a 152 x 152 UB23 for column section and assembled to form a cruciform arrangement. The end-plate extended on the tension side only with a thickness of 15 mm and was bolted to the column with two M16 bolts on each side of tension flange and two M16 bolts in the compression region.

The limited experimental evidence shows that the buckling plates are a very effective means of increasing the stiffness and the yield load for an extended end-plate connection by at least 20%. The ultimate mode of failure and the failure load for each specimen were similar. The two M-Φ curves provided exhibit a nonlinear behaviour throughout the loading.

Zoetemeijer and Munter [4.36] carried out an experimental investigation on four unstiffened extended end-plate connections to get an indication of the influence of the longitudinal stress in the column flange on the tension side of the connection. All test specimens have the same configuration (i.e. bolts and end-plate) and were tested in cruciform arrangement. The bolt configuration is chosen in such a way that yielding of the column at the tension side is the determining failure mechanism of the connection. The test frame consisted of a vertical column stub HE240A connected to a horizontal IPE400 beams.

Test 1 was carried out without any axial load in the column and served as a reference. An axial load of 1000 kN was applied and maintained during testing in test 2. This load gave a longitudinal stress of 135 N/mm^2 in the flanges. As the moment-rotation curves of these two tests were very similar to each other, tests three and four were tested under axial load with a bending moment in the column obtained by giving the hydraulic jack an eccentricity of 70 mm. On the basis of this test programme the authors concluded that:

i) The axial compressive load which produces a longitudinal stress of 135 N/mm^2 in the column flanges has negligible effect on M-Φ behaviour, even at extremely large rotations and does not reduce the moment capacity of the connection (i.e. test 2).

ii) From the other tests in which the column load was applied eccentrically it would appear that longidutinal stresses up to 170 N/mm^2 (about 70% of the yield stress) have negligible effects on M-Φ behaviour.

iii) Above a longitudinal stress of 170 N/mm^2 the connection stiffness at the ultimate design moment reduces by a factor of 2.

iv) The rotation capacity of the connection remains constant at a longitudinal stress of 215 N/mm².

Moment-rotation curves for tested specimens are available on reference [4.36].

Tong [4.37] conducted several tests on flush and extended end-plate connections at Hatfield Polytechnic, in 1985, to provide data on moment-rotation characteristics for bolted connections. In total 18 specimens were tested, 12 of which were extended end-plate connections. Actual forces in the tension bolts and development of prying forces were also investigated. Beam sections, column sections, bolt diameters and bolt grades were chosen according to the reply of the survey of the structural steelwork industry [4.38, 4.39].

Stiffened isolated end-plates extended on one side or on two sides of the beam flanges were investigated by Zandonini and Zanon [4.2, 4.40]. In total 10 test specimens were carried out in two groups of five. Within the same group of connection only the plate thickness was varied, ranging from 12 mm to 25 mm, allowing different ratios between the plate and bolts stiffness to be obtained. The specimens were subjected to loading and unloading cycles (but no reversal of loading) applied as a horizontal force to IPE300 beam stub. Bolts M20 grade 8.8 were used in all the connections. The experimental results show that the moment capacity for all the connections tested was greater than the plastic moment of the beam, based on the experimental yield strength and the actual dimensions of the cross section, and the ultimate strength increased with the end-plate thickness. However, the rotation capacity of the connection was found on the contrary adversely affected by the increase of the end-plate thickness. This parameter decreases from 0.08 to 0.02 radians for end-plates extended on tension side only and from 0.05 to 0.01 radians for end-plates extended on both sides of the beam. A comparison between the curves related to the corresponding specimens of the two series shows that the presence of the extension of the end-plate on the compression side has a limited influence on the connection behaviour and strength. Zandonini and Zanon [4.40] concluded from these test specimens that the moment-rotation curves do have a very similar shape, despite the differences in strength and ductility, characterised by an almost elastic response up to a moment value ranging from 40% (thin end-plate) to the 60% (thick end-plate) of the ultimate moment capacity followed by a behaviour remarkably non linear.

On the same basis as the work carried out by Tong [4.37], Prescott [4.1] extended the data available on end-plate connections by testing another 10 isolated connections and two full scale frames. Only on four test specimens were extended end-plate connections used, with stiffened column webs. The type of end-plate (flush or extended), end-plate thickness and bolt preload were the key parameters of the experimental investigation.

In 1987 for his PhD thesis, Davison [4.41] conducted a series of tests on beam-to-column connections. Extended end-plate connections were used in six test specimens, two of which were connected about the minor axis and are not included in this study. The cruciform arrangement comprised a 254 x 102 UB22 beam and a 152 x 152 UC23 column. A 16 mm diameter bolt was used to fasten the 15 mm extended end-plate to the column flanges which were stiffened with three column web stiffeners at the tension region and one at the compression region. Two specimens were tested to examine the effect of lack of fit on connections incorporating untorqued bolts with a deliberately distorted end-plate. Only on one test were HSFG bolts used instead of grade 8.8 bolts. The examination of test results show that the behaviour of the connection was nonlinear throughout, but with a knee occurring at about two-thirds of the beam's plastic moment. Lack of fit did not adversely affect the connection's performance, where the friction between the plate is not important and does not affect the ultimate tensile capacity. However, moment-rotation curves indicate that connection with HSFG bolt has a greater initial stiffness than the 8.8 bolt connection.

Chakrabarti [4.42] investigated experimentally the behaviour of the unstiffened bolted flush and extended end-plate connections that were used in full scale test frame at BRE in collaboration with Hatfield Polytechnic and Sheffield University. The test specimen comprised two beams 254 x 102 UB28 and a column 152 x 152 UC37 to form a cruciform arrangement. Thus, two identical extended end-plate connections with a 385 x 154 x 15 mm end-plate were tested. The beam to column joints were bolted with hand tightened nuts and bolts. For this type of unstiffened connection, the failure mode was due to excessive yielding of the column flanges and compression in the column web. From the experimental investigation, he concluded that for these connections the point of rotation is at the lower edge of the end-plate and not opposite the beam compression flange. The extended end-plate transmitted 47.4% of the beam yield moment at yield and 98.9% at failure.

Ten of the twenty two beam-to-column unstiffened connections tested by Janss et al [4.43] at Liege University, Belgium in 1987 were extended end-plate connections. In four specimens, the connections were tested under cyclic loading and they are not included in this study. The six specimens subjected to static loading included end-plate thickness varying from 15 mm to 20 mm. The load was applied at the end of the cantilever beam while the column was subjected to an additional axial load in two test specimens.

The relative rotation of the joint, as well as the shear flexibility of the column web panel were measured. From the test results the relevant conclusions are summarised as follows:

i) The effect of the axial force in the column, on the shear flexibility of the column web panel can be disregarded if it does not exceed half the squash load.

ii) The initial stiffness of the joint seems just be little influenced by the axial force.

iii) The initial stiffness of the connection depends on the dimensions of the column flange. The weaker the column flange, the more marked the nonlinear behaviour of the moment-rotation curve which is due to yielding of the compressed zone of the column web.

4.3.2 Analysis of moment-rotation characteristics

In the previous section, complete moment-rotation curves for connections of varying dimensions, carried out during the last three decades under static loading conditions, were reported. Using this static data bank, the geometric parameters and material properties of the joint that most significantly affect the moment-rotation relationship of the connection will be identified in this section.

4.3.2.1 *Effect of end-plate thickness*

Examination of experimental M-Φ curves show that the behaviour of the connection is sensitive to the thickness of the end-plate. Comparison of moment-rotation curves where the only variable in each set is the end-plate thickness are shown in Figure 4.3a and 4.3b. Figure 4.3a corresponds to tests 7 and 8 of reference [4.37] for extended end-plate connections over the range of thicknesses 15 and 20 mm. The moment-rotation curves are almost identical up to a moment of approximately 35% of the maximum applied moment, then a large increase in rotation occurs. The rotation up to this level of moment is mainly due to bolt stretching and plate bending, then the curves separate according to the end-plate thickness. Thick end-plates produce little if any rotation due to end-plate bending and column deformation. As the applied loads for these tests were prematurely stopped due to the hydraulic jack reaching its maximum capacity, the comparison of moment-capacity between these two connections cannot be made. Similar results are shown in Figures 4.3b from reference [4.1] where the end-plate thicknesses are respectively 20 and 25 mm for tests 22 and 23. The separation of the curves occurs at about 30% of the connection ultimate moment. Test 23 was not completed and consequently the failure stage was not attained.

As a result of several comparisons of moment-rotation curves which are not included here, the end-plate thickness is obviously a highly influential parameter for end-plate connections. Thick end-plates form significantly stiff connections and produce less rotation for the same value of moment. Hence they may seriously restrict the rotation capacity of the connection, i.e. its ability to deform plastically. Experimental results of Bailey [4.26] have shown that thick end-plates may improve the initial stiffness of the connection but will not necessarily enhance its moment capacity. But with relatively thin end-plates, bending of the end-plate is important which in turn transfers more bending in tension bolts lowering their moment capacity. It has been previously shown that the end-plate thickness affects not only the connection stiffness and failure mode but it affects as well the column flange behaviour.

4.3.2.2 *Effect of beam depth*

Similar to the flush end-plate connections, the depth of the beam and the depth of the extended end-plate are directly related. Hence, a deeper beam section requires a deeper end-plate connection. Test C6 of Surtees and Mann [4.8] investigated the problems associated with deeper beam sections with a limitation that four bolts were used to resist tension force. As beam depth increased, the tensile force from the beam became very large and consequently it was difficult to transfer such highly concentrated forces through the connection material. Although column web stiffeners opposite both beam flanges were used in this test, the connection failed in the weld without reaching the plastic moment of the connected beam. However, the failure mode for the other five unstiffened connections with shallow beam sections was at a moment well beyond the plastic moment of the connected beam. Unfortunately a direct comparison on the M-Φ curves is not possible due to the fact that the connection with a deeper beam section was provided with column web stiffeners.

4.3.2.3 *Effect of end-plate depth*

As previously mentioned, the depth of the end-plate is related to the beam section. A deep beam section requires a deeper and thicker end-plate connection. This is shown in tests 07 and 010 of Janss et al. [4.43].

The presence of the extension of end-plate on the compression side is shown in Figure 4.4 for a family of M-Φ curves (denoted EPB) tested by Zandonini and Zanon [4.2] with end-plate thickness varying from 15 to 25 mm. The corresponding specimens (EP) had end-plates extended on the tension side only. Comparison between the curves of the two series indicates that the connections behave in a very similar manner, i.e. almost identical shape of M-Φ curves, and the failure modes are practically the same.

As the end-plate depth is related to the beam section and as the extension of the end-plate on the tension side does not affect significantly the connection behaviour and strength, it would be possible to leave out such parameter in predicting the M-Φ relationship provided the beam depth parameter is already taken into account.

4.3.2.4 *Effect of bolt number and arrangement*

Moment-rotation curves for tests 4 and 13 of reference [4.37] differed from each other by having 5 mm smaller vertical bolt pitch (i.e. distance between the tension bolt rows) and also a reduction in bolt centres by 20 mm. The curves are plotted in Figure 4.5. Such reduction in bolt centre distance resulted in an increase in the connection stiffness and in the ultimate strength.

Experiments [4.1, 4.9, 4.37] have shown that the bolt location is critical in the performance of the connection. Normally bolts should be placed as close as possible to the tension flange in order to reduce the prying forces and to make the connection stiffer. There are certain practical limitations, the bolt positions being limited by clearances to nuts, washers and tightening tools. The bolt number and arrangement greatly affect the bending behaviour of the connection in addition to simply connecting the column flange and the end-plate. As such, the effect of the bolt location on the bending behaviour must be taken into account as far as possible to accurately model the connection behaviour.

4.3.2.5 *Effect of bolt preload*

From the experimental results on isolated tee-stubs, Packer and Morris [4.4] reported that the amount of bolt prestress does not affect the collapse mechanism or the yield load, and this is likely to be generally true except for the extreme cases where the tee-stubs or end-plate shape is very distorted due to the fabrication process [4.4].

However, as for the flush end-plate connections described in section 3.3.2.4, the elastic stiffness increased as the preload force of bolts increased for connections with the same end-plate thickness. Figure 4.6 shows a family of M-Φ curves tested by Tong [4.37] where the only variable in each set is the bolt preload value. The two

12 mm extended end-plate connections of Figure 4.6a, in which the bolts in test 4 were hand tightened (about 25 kN) and in test 15 were preloaded to proof load of the bolts (135 kN), clearly indicates the preload effect on connection behaviour. Similar results are shown in Figure 4.6b for end-plate of 20mm thickness where the increase of connection stiffness is much higher than for the 12mm end-plate connections. The increase of the elastic stiffness (especially initial connection stiffness) is related to the ratio of bolt to plate stiffness. An interesting result is shown in Figure 4.6a where the initial stiffness of the 12 mm extended end-plate connection of test 4 with bolts preloaded to 135 kN is higher than the corresponding values of the 20 mm thick with bolts preloaded to about 25 kN.

Figure 4.6c shows similar differences in elastic stiffness, for tests 22 and 27 of reference [4.1] where the end-plate was 20 mm thick. Therefore, based on experimental evidence on extended end-plate connections [4.1, 4.21, 4.37], the stiffness of the connection in the range of working loads is greatly influenced by the bolt preload force but the failure load of the end-plate connection appears to be independent of the magnitude of the pretensioning force. Hence, bolt preload factor is an important parameter for predicting moment-rotation relationships.

4.3.2.6 *Effect of column section*

Certainly, the M-Φ curve relationship is controlled by the portion of the column flange situated between the position of stiffeners. Since the same end-plate and beam sizes were used for all tests of reference [4.4], the moment-rotation characteristics for the three joint tests can be compared to show the effect of column strength upon this behaviour. This is shown in Figure 4.7a. The measured column flange thickness covered the range of 6.5 mm, 9.5 mm and 11.9 mm respectively for tests J2, J5 and J1, to represent a column flange stiffness of lower, similar and greater stiffness than the end-plate (15 mm). The total rotation achieved for all the specimens was approximately the same because the beams can only sustain a certain amount of rotation before local beam compression flange buckling occurs near the joint. This rotation is achieved at a lower applied moment for joint J2 (relatively more flexible) and at a higher moment for stiffer joint as in test specimen J1. Also, comparison of these M-Φ curves revealed that a reduction in column flange thickness reduced the initial connection stiffness appreciably. It also changed the shape of the M-Φ curve; for an 11.9 mm flange thickness the initial portion up to about 45% of capacity was essentially linear whereas for a 6.5 mm flange thickness the curve was nonlinear from a loading which corresponds to about 30% of the capacity of the connection. Test results of specimen J2 shows that the use of too thin column flange prevents the attainment of full beam strength.

Similar conclusions have been shown experimentally by Grundy et al [4.11]. Column flange deformation does influence connection behaviour and should be taken into account in predicting the M-Φ relationship for unstiffened connections.

However, column flange thickness has negligible effect if column web stiffeners are provided. The M-Φ behaviour of tests 11 and 14 of reference [4.37] is plotted in Figure 4.7b. Test 11 was conducted with a lighter column stub (254 x 254 UC73) and test 14 with a heavy column stub (254 x 254 UC132). As the end-plate was thick (25 mm) the failure mode was by fracture of the inner bolt row in test 14, however failure stage was not reached in test 11 (i.e. test 11 prematurely stopped). Experimental investigation [4.37] has shown that for end-plate connections with stiffened column flange, there is no significant increase in strength by having the column flange thicker than the end-plate.

4.3.2.7 *Effect of column stiffeners*

It is important to classify the end-plate connections into two distinct categories:

i) Connections stiffened in the column web; and

ii) Connections not stiffened in the column web.

Experimental investigations have shown that connections falling in the first category did not experience any failure in the column flange or web. However, the stiffened column caused significant increase in the connection stiffness. Comparison of M-Φ curves for tests 6 and 11 of reference [4.37] shown in Figure 4.8a confirms that the presence of horizontal column web stiffeners opposite both beam flanges increases the stiffness of the connection. This increase is not limited to working load level but over the entire loading. Although, in test 11 the failure stage was not reached, the comparison can still be made. The column stiffeners not only increase the connection stiffness but it increases its moment capacity as well as the extent of the initial portion of the M-Φ curve. Similar behaviour was observed by providing flange backing plates as an alternative of column web stiffeners in references [4.12, 4.15]. However, experimental evidence on the effect of column stiffening appears contradictory; in tests J2 and J3 of reference [4.4] shown in Figure 4.8b, the presence of column web stiffeners enabled the connection to develop the full plastic moment at large rotations but only increased joint stiffness in the upper region of the M-Φ curve.

4.3.2.8 *Effect of material yield stress*

Beyond the yield point, the accuracy of the theoretical moment-rotation relationships depends on the actual yield stress of the steel material used [4.1, 4.25, 4.37]. In all collected specimens, the actual yield stress of the steel material determined from the tensile test is normally higher than the value quoted in BS5950 [4.44]. Figure 4.9 shows a comparison of predicted and experimental curves for test 4 carried out by Tong [4.37] for 12 mm thick extended end-plate connections with different value of end-plate and column flange yield stress. The curve labelled (a) was obtained using a value of yield stress of 240 N/mm^2, whereas curve (b) included the weld element and its associated enhanced yield stress value. It is clear that curve (b) predicts more accurately the behaviour of the joint specially beyond the elastic range.

As previously discussed for the flush end-plate connections (section 3.2.7), as the load increases the connection stiffness reduces due to yielding of one or more of the constituents of the joint. Thus the softening of the connection is not due to only the yielding of the end-plate. Therefore, it is important to incorporate into the analytical model the actual yield stress of the steel material in the end-plate, column and beam in order to improve the predicted behaviour of the connection beyond the elastic range.

4.3.2.9 *Effect of shear*

The influence of shear force on the behaviour of the connections was studied in references [4.26, 4.37]. For comparison of the M-Φ curves, Tong [4.37] tested 2 pairs of identical connections. Tests 7 and 9 were identical extended end-plate connections of thickness 15 mm. The other identical pair of connections, tests 8 and 10, were 20 mm thick. Tests 7 and 8 were conducted under the pure moment loading. Tests 9 and 10 were subjected to shear and moment. From these results, shown in Figure 4.10, it was concluded that the effect of shear force was negligible on the connection behaviour. However, no conclusion can be drawn from these tests on the ultimate moments as tests 7 and 8 were stopped prematurely.

4.3.2.10 *Effect of axial compressive loads*

The influence of compressive axial load on the moment-rotation response of connections was studied in earlier investigations [4.4, 4.21, 4.27, 4.36] and recently by Janss et al [4.43].

In all tested specimens of references [4.21, 4.27] a compressive axial load of approximately one third the nominal yield axial load was applied to the top of the column through a hydraulic jack-load cell combination. Unfortunately no specimen was tested without axial compressive load, therefore a direct comparison between M-Φ curves was not possible.

Some of the evidence appears contradictory, tests 1 and 2 of reference [4.36] show that an axial compressive load which produces a longitudinal stress of 135 N/mm^2 in the column flanges has negligible effect on M-Φ behaviour, even at extremely large rotations. Nor does it reduce its moment capacity. From the other tests in which the column load was applied eccentrically it would appear that longitudinal stresses up to 170 N/mm^2 have negligible effect. However for test J4 of reference [4.4] the presence of a compressive axial load in the column of 150 kN (35% of its working load) affected the yield moment of the column flange so that the linear portion of the M-Φ curve is reduced. This observation is in accordance with Janss et al's conclusions [4.43] but in contradiction with Zoetemeijer and Munter [4.36]. The tests 01, 04 and 07 of reference [4.43] were identical except a value of 300 kN, 700 kN and zero axial load were applied respectively. From these latter test results, it seems that the initial joint stiffness to be little influenced by the axial force. Janss et al [4.43] concluded that the axial force has no sensible effect on the yield shear force in the range of $\sigma < 130$ N/mm^2 (i.e. $\sigma = 0.5\ F_y$).

The comparison of moment-rotation relationships between the two tests J2 and J4 are given in Figure 4.11 (6.5 mm column flange thickness and 15 mm end-plate thickness). It can be seen that initial stiffness up to about 30% of the capacity of the connection to be unaffected by the presence of the axial force. Then, the curves start to separate with the tendency to become more flexible (the M-Φ curve corresponding to test J4). As mentioned before, yielding of the column flange is also affected and consequently the linear portion of the M-Φ curve of test J4 is reduced approximately from 32% to 25% of the corresponding failure moment of the connection.

The limited experimental investigations carried out has indicated that the presence of axial compressive load does affect the yield moment of the column flange and the connection stiffness beyond this point, but before this parameter is included in the analytical representation of the M-Φ behaviour a more exhaustive investigation is required.

As a result of this parametric analysis with reference to Figure 4.1 the major parameters that affect the extended end-plate connections are:

i) End-plate thickness, T_p,

ii) Beam depth, D_b,

iii) Bolt positions; this includes the horizontal gauge distance of the bolts, G, and vertical bolt pitch, P,

iv) Bolt preload, P_l, and proof load, P_f,

v) Column flange thickness, T_{fc} unless the connection is attached to rigid supports or stiffened column flanges, or for a connection such as beam splices, in which a column is not involved,

vi) Whether the end-plate is connected to stiffened or unstiffened column, and

vii) Material properties.

Therefore, to achieve a good analytical representation of moment-rotation relationship, the above major connection parameters should be included in the model.

4.4 Analytical representation of moment-rotation curves

The total research of extended end-plate connections is divided into two separate but related areas: analytical and experimental. So far, experimental study has been investigated. Analytical results will now be compared with the results of the experimental data, to investigate verification of the analytical methods.

4.4.1 Frye and Morris's model

As described in the previous chapters, Frye and Morris [4.45] developed a standard moment-rotation curve for the end-plate connection by using a polynomial approximation to fit the test data. The standard curve employed the use of size factors which related to selected dimensions of the connection.

Frye and Morris [4.45] differentiate between the end-plate connection without column stiffeners and the connection with column stiffeners. However no distinction was made between the flush and the extended end-plate connections, as earlier described in Chapter Three.

i) The prediction equation of connection rotation for an extended end-plate connection without column stiffeners is given by:

$$\Phi = 1.83 \times 10^{-3}(KM) - 1.04 \times 10^{-4}(KM)^3 + 6.38 \times 10^{-6}(KM)^5 \quad (4.1)$$

where

$$K = L_b^{-2.4} \cdot T_p^{-0.4} \cdot T_{fc}^{-1.5}$$

ii) For an extended end-plate connection with column web stiffeners, the connection rotation is given by:

$$\Phi = 1.79 \times 10^{-3}(KM) + 1.76 \times 10^{-4}(KM)^3 + 2.04 \times 10^{-4}(KM)^5 \quad (4.2)$$

where

$$K = L_b^{-2.4} \cdot T_p^{-0.6}$$

The parameters L_b, T_p and T_{fc} are defined in Figure 4.1 and are in inches, the moment M is in kips-in and the rotation Φ is in radians.

Similar to the flush end-plate connections, to get a better approximation of the M-Φ true behaviour of the connection, L_b should be taken as the depth D_b of the connected beam [4.23].

Equations (4.1) and (4.2) are given in metric units in section 3.4.1. of Chapter Three.

4.4.2 Krishnamurthy, Huang, Jeffrey and Avery's model

In 1979, Krishnamurthy et al [4.46] attempted to determine the rotation of end-plate moment connections, from a computer analysis based on a two dimensional finite element model and resulting empirical equations. The two dimensional analysis is an in-plane stress analysis, in a plane parallel to the beam web. A correlation factor between two and three dimensional model was developed by Krishnamurthy and Graddy [4.47]. The element permitted non linear material properties and the idealised stress-strain behaviour of steel was represented as the elastic-perfectly-plastic bilinear variation. The finite element analysis [4.46] assumed the column flange as completely rigid. However, it has been shown in section 4.3.2.6 that the column flange deformation does influence connection behaviour and should be taken on consideration. Therefore the model should be applied to beam-column end-plate connections with very thick or stiffened flanges.

It appears that the expression was obtained by applying multiple regression analysis on the results of 168 connections analysed using the finite element technique. The proposed power function to represent the moment-rotation curves of an extended end-plate connection was expressed as:

$$\Phi = \frac{\beta \ \mu \ P_f^{2.03} \ M^{1.58}}{a_b^{0.36} \ t_p^{1.38}} \tag{4.3}$$

in which

β is a function of the beam dimensions designated as the beam factor and is given by:

$$\beta = \frac{0.0056 \ b_f^{0.61} \ t_{fb}^{1.03}}{d^{1.30} \ t_{wb}^{0.26} \ S_x^{1.58}} \tag{4.4}$$

μ is a function of material properties and designated as the material factor,

$$\mu = \frac{1.0}{F_{yp}^{0.38} \ F_{by}^{1.20}} \tag{4.5}$$

where

P_f is the actual bolt distance,

a_b is the actual bolt area per row,

t_p is the end-plate thickness,

b_f is the beam flange width,

t_{fb} is the beam flange thickness,

d is the beam depth,

t_{wb} is the beam web thickness,

S_x is the section modulus of the beam,

F_{yp} is the yield stress of the plate material, and

F_{by} is the yield stress of the bolt material.

Fifteen beam butt plate splice tests were performed to check the validity of the analysis, but no details of the test specimens or results were given. Comparisons [4.46] of calculated and measured results showed that the finite element model were much more flexible that the true behaviour due to the omission of bolt heads and fillet welds between the beam flange and the end-plate. As a result of this, in the actual bolt distance, P_f, in equation (4.3), the omitted details were accounted for by the use of an effective bolt distance, P_e, given by:

$$P_e = P_f - d_b/4 - 0.707\, W_s \tag{4.6}$$

where

d_b is the nominal bolt diameter, and

W_s is the weld size.

In the developed model, the bolts were assumed pretensioned to approximately 0.8 times the nominal bolt yield stress or 0.7 times the bolt ultimate strength.

In the above equations, original notation is used throughout the text to avoid any possible confusion and the connection parameter dimensions are in inches, the material properties are in ksi and the moment is in kip.in.

4.4.3 Tarpy and Cardinal's model

Tarpy and Cardinal [4.21] presented an analytical study of extended end-plate unstiffened beam to column connections. The finite element model used was based on a linear elastic model. A parametric study was conducted on 97 different connections generated from the finite element program, from which an M-Φ equation has been developed using multiple linear regression technique.

The moment-rotation relationship was expressed as:

$$M = \frac{2.65 \times 10^{4} . P^{0.65} . TF^{1.81} . TE^{1.40} . D^{1.32}}{FW^{0.58} . G^{1.59}} \Phi^{0.76} \qquad (4.7)$$

and the moment capacity was given by:

$$M = 1.65 \; F_y^{0.64} . TE^{0.92} . TF^{0.87} . G^{0.14} . D^{0.72} . P^{-0.22} \qquad (4.8)$$

The parameters used in equation 4.7 and 4.8 are defined as follows:

D is the beam depth,

G is the gauge distance,

P is the pitch distance,

TF is the column flange thickness,

TE is the end-plate thickness,

FW is the flange width,

F_y is the yield stress of column and end plate material,

M is the applied beam end moment, and

Φ is the connection rotation.

In the above equations, the connection parameters are in inches, the moment is in kips. ft and the connection rotation in radians. Original notation is used in equations (4.7) and (4.8).

4.4.4 Yee and Melchers's model

Yee and Melchers [4.48] developed a physically based mathematical model to predict the moment-rotation relationship of bolted extended end-plate eave connections. The four parameter model takes into account the possible failure modes and the deformation characteristics of the connection elements.

The moment-rotation relationship is expressed as:

$$M = M_p \left\{ 1 - \exp\left[\frac{-(K_i - K_p + C\Phi)\ \Phi}{M_p} \right] \right\} + K_p\ \Phi \qquad (4.9)$$

The four main parameters of the model are:

i) *The plastic moment capacity of the connection, M_p* :

For stiffened connection, M_p is the plastic moment capacity of the weakest adjoining section (beam or column). However, for unstiffened connections, this value depends on the mode of failure of the weakest connection element which were previously described in section 4.2 Values of M_p are given in reference [4.48] for the different failure modes for an unstiffened connection.

ii) *The initial elastic stiffness of the connection,* K_i:

This value depends on the deformation of the individual connection elements, i.e. end-plate and column flange flexure for both stiffened and unstiffened columns, bolt extension at the beam tension flange level, shear deformation of the column web panel and compression deformation of the column web which is not negligible for unstiffened connections. The other alternative, to stiffen the column flange by a doubler plate, and the effect of bolt pretensioning were also included in formulating the initial stiffness K_i.

iii) *The strain-hardening stiffness of the connection, K_p*:

In unstiffened connections, the strain-hardening stiffness is considered important for two modes of failure: shear yielding of the web and web buckling in the compression zone at flange level.

Values of K_i and K_p are expressed in mathematical forms in reference [4.48].

iv) *The empirical coefficient, C*:

The constant C is determined by calibrating the model, equ. (4.9), against experimental M-Φ curves obtained from 16 tests conducted by Yee [4.49] and depends on the type of connection. The adopted values of parameter C is zero for stiffened connection with snug tightened bolts, 3.5 for stiffened connection with pretensioned bolts and 1.5 for unstiffened connections.

4.5 Comparison of experimental and analytical moment-rotation curves

4.5.1 Extended end-plates with column stiffeners

4.5.1.1 *Frye and Morris's prediction equation*

Frye and Morris [4.45] considered two prediction equations according to the condition of the column flange. The prediction equation of the connection rotation for an extended end-plate with column web stiffeners given by equ. (4.2) was based on 18 experimental curves. This data base consisted of 13 flush end-plates of Ostrander [4.50], and on only five extended end-plate test results, in which four connections were tested by Sherbourne [4.10] and one by Johnson et al [4.22]. As a result of this and as mentioned in section 4.4.1, Frye and Morris make no distinction between flush and extended end-plate connections.

The accuracy of the standardisation procedure is illustrated by Figures 4.12 through 4.45, which show the moment-rotation curves generated by the prediction equations and the corresponding experimentally obtained curves for stiffened extended end-plate connections. In total a possible of four analytical M-Φ curves are shown for each test results where data is available. Three of which are based on Frye and Morris model [4.45] and one on Krishnamurthy et al model [4.46].

The Frye and Morris model (with the parameter L_b as the distance between extreme bolt centres is based on equ. (4.2). The Frye and Morris model with the parameter D_b as the depth of the connected beam as suggested by Goverdhan [4.23] is also based on equ. (4.2). Finally the present modified Frye and Morris model, suggested by the present author, with the parameter D_b as the beam depth, this is based on the following prediction equation:

$$\Phi = 1.79 \times 10^{-3}(KM)^{1} + 1.76 \times 10^{-4}(KM)^{3} + 2.04 \times 10^{-4}(KM)^{5} \qquad (4.10)$$

where

$$K = D_b^{-2.6} \cdot T_p^{-0.6}$$

The difference between the latter prediction equation (i.e. equ. (4.10)) and the original Frye and Morris prediction equation (i.e. equ. (4.2)) is that the parameter L_b in equ. (4.2) becomes D_b in equ. (4.10) and with a power of -2.6 instead of -2.4 as given in references [4.23, 4.45] to improve the connection stiffness and consequently a better approximation as shown in Figures 4.12 to 4.45.

Figures 4.12 to 4.16 show comparison between predicted M-Φ curves against the data used by the investigators [4.45] to develop their own prediction equation. Certainly as shown in the figures, the maximum deviation of predicted curves from experimental curves is not within 6% as claimed by Frye and Morris.

In all test results the Frye and Morris predicted curves do not show any presence of reverse curvature in the initial region. The reverse curvature implies that the connection stiffens with increased moments. In all cases the predicted curves are extremely flexible. Obviously as shown by Goverdhan [4.23] taking L_b parameter in equ. (4.2) as the beam depth will increase the connection stiffness but that does not lead to a reasonable approximation of the true connection behaviour. The author felt that the connection stiffness is increased by reducing the power value of the beam depth parameter and therefore equ. (4.10) should be used to predict the moment-rotation curve for end-plate connections with column stiffeners.

Moment-rotation curves in Figures 4.12 to 4.16 show the behaviour of extended end-plate connections tested by Johnson [4.22] and Sherbourne [4.10]. The modified Frye and Morris curves predicted by equ. (4.10) does not approximate closely the connection behaviour despite the fact it is predicted on the flexible side and the connection stiffness is higher than the one given by equ. (4.2) over the entire range of loading. This may be due to the fact that in all these connections, five rows of bolts were used and the test specimens were fabricated from ordinary mild steel to BS 15-1948.

Due to the compexity of the connection behaviour itself, prediction of M-Φ curves for such connection is extremely difficult to develop. In Figures 4.36 to 4.45 which represent the experimental results of tests carried out by Bailey [4.26], despite the fact that the connections on the left and on the right of the cruciform test type were identical, under the same conditions of loading and from the same test rig, the corresponding M-Φ curves do not show similar behaviour of the connection especially for test specimens B6, B8, C11 and C13. Thus, a conclusion can be drawn for these results is that using such data base, one would not expect to achieve a perfect fit over the entire range of loading no matter what techniques are used to develop these prediction equations.

In all cases, equation (4.10) underestimates the connection stiffness except for tests J3 of Packer, J4 of Moore and Sims, JT/13 of Davison and C11 of Bailey shown in Figures 4.18, 4.19, 4.35 and 4.43 respectively. The ratio of the end-plate thickness to the beam depth for these connections is 0.0590 except for test C11 of Bailey which was 0.0606. In general the equation (equ. (4.10)) predicts reasonably well the connection stiffness. At high moment especially for connection with high bolt preload such as in Figures 4.30, 4.33 and 4.34, where the bolt preload was equal or greater than 100 kN, the equation does not predict closely the connection behaviour.

This is due to the fact that the model does not take into consideration of the effect of the bolt preload. Equation (4.10) gives poor M-Φ curve representations for connections which failed by bolt fracture as shown in Figures 4.13, 4.14, 4.24, 4.25, 4.28, 4.30, 4.31, 4.33 and 4.34.

4.5.1.2 *Krishnamurthy et al prediction equation*

As described in section 4.4.2 of this chapter, the model is based on a finite element analysis. The investigators [4.47] have stated that the analysis applies only to very thick or stiffened column flanges or to symmetrical end-plate connections. For unsymmetric longitudinal splices and for unstiffened beam-to-column connections, the rotation contributions by the two halves of the connection must be calculated separately and added [4.47]. As the column flange behaviour was not included in any form nor at any stage of the derivation of the prediction equation, the present author concluded that this model is not suitable for unstiffened extended end-plate connections.

Figures 4.12 to 4.32 show predicted curves developed by Kirshnamurthy et al against experimental curves (where data is available) for a wide range of end-plate thickness, beam and column sections and bolt preloads. In all cases checked, the predicted curves are extremely stiff except for tests A3, B1 and B2 of Sherbourne [4.10] shown in Figures 4.14 to 4.16 respectively. The curves are almost straight lines over the entire range of loading. Even at very low moments the model overestimates considerably the initial tangent stiffness. As the moment increases, the predicted curve deviates considerably from the true connection behaviour. Since this equation does not approximate the experimental data at any stage of loading it is recommended that the equation should not be used for predicting the moment-rotation relationship.

4.5.2 Extended end-plate connections to unstiffened column

4.5.2.1 *Frye and Morris prediction equation*

The prediction equation of the connection rotation for extended end-plate without column web stiffeners given by equ. (4.1) was based on 12 experimental test specimens. This data base consisted mainly on flush end-plate connections tested by Ostrander [4.50]. Only one extended end-plate connection carried out by Sherbourne [4.10] was used and therefore as stated earlier Frye and Morris [4.45] make no distinction between the two different type of connections.

The four predicted curves are presented against the experimental curve in Figures 4.46 to 4.58 for each test specimen. Two predicted curves are based on Frye and Morris's prediction equation (equ. (4.1)). The only difference between the two is that the parameter L_b in equ. (4.1) is firstly taken as the distance between the extreme bolt centres and secondly as D_b, the connected beam depth as suggested by Goverdhan [4.33]. Again the author felt that it is necessary to increase the connection stiffness by modifying the Frye and Morris prediction equation. The connection rotation is given by:

$$\Phi = 1.83 \times 10^{-3}(kM) - 1.04 \times 10^{-4}\ (kN)^{3} + 6.38 \times 10^{-6}\ (kM)^{5} \qquad (4.11)$$

where

$$K = D_b^{-2.6}\ .\ T_p^{-0.4}\ .\ T_{fc}^{-1.5}$$

Figures 4.46 to 4.58 show that there is not much improvement by taking the parameter L_b of equ. (4.1) as the beam depth in predicting the M-Φ curves. In both cases the predicted curves are far more flexible than the experimental curves and do not represent in any case and at any stage the true connection behaviour. However, the modified Frye and Morris prediction equation (i.e. equ. (4.11)) approximates reasonably well the experimental curve up to rotation of about 0.02

radians after which deviation is substantial. It has been shown in section 2.2 of Chapter Two that the maximum rotational deformation for a beam-to-column connection in a frame designed to meet the recommended deflection limits would be 0.023 radians for steel grade 43. Therefore for an extended end-plate connection within a frame, the maximum connection rotation expected should be less than 0.023 radians and consequently equ. (4.11) can be used.

Again as it was the case for stiffened connection, for connection A1 tested by Sherbourne [4.10] and shown in Figure 4.46, the modified Frye and Morris curve predicted by equ. (4.11) does not approximate closely to the experimental curve. The predicted connection stiffness is much better though than the one given by equ. (4.1) over the entire range of loading. The predicted M-Φ curves are much more flexible than the experimental curve, this may be due to one of the following reasons: five rows of bolts were used in this connection, the test specimens were fabricated from ordinary mild steel to BS 15-1948 and/or the end-plate is too thick (32 mm). The main disadvantage of equ. (4.11) is that erratic change in the tangent stiffness may occur in the early stage of loading as is the case in Figures 4.46 and 4.47. This is due to the presence of the negative sign in the polynomial equation of Frye and Morris. Such curves should be avoided in frame analysis if the tangential stiffness is required. To overcome this problem, the initial part of the curve where reverse curvature appears can be approximated by a straight line as shown in Figure 4.59 for connection C1 tested by Mann [4.25].

The predicted curves shown in Figures 4.47, 4.48, 4.49, 4.50, 4.51, 4.52, 4.57 and 4.58 approximates the experimental curves reasonably well up to rotation of about 0.02 radians. The ratio of the column flange thickness to the beam depth for these connections is greater or equal to 0.045. However, the prediction equation gives poor results for ratio of column flange thickness to beam depth less than 0.040.

4.5.2.2 *Tarpy and Cardinal's prediction equation*

The prediction equation was generated from a finite element analysis based on a linear elastic model. Tarpy and Cardinal [4.21] provided also an equation to predict the moment capacity of the connection. The model does not take into consideration in any form of the bolt preloading. The material yield stress is taken into account only for the determination of the moment capacity of the connection. It is not clear in equ. (4.8) whether the parameter F_y represents the yield stress of the column or of the end-plate material.

Families of experimental and predicted M-Φ curves are shown in Figures 4.46 to 4.58 for extended end-plate connections to unstiffened columns. In all cases checked the predicted curve is almost a straight line and simulates the connection behaviour in a very stiff manner. The predicted moment capacity of the connection is also indicated in the plots by the notation M_c for each test specimen. This moment capacity is in considerable error. Comparison of experimental and predicted moment capacity of the connection, Mc_{pred}/Mc_{exp}, varies between 0.20 and 0.50 except for test 6 carried out by Tong [4.37] shown in Figure 4.58 which is of the order 0.85.

4.5.3 Yee and Melchers's prediction equation

The non-linear mathematical model is suitable only for bolted end-plate connections that are used for eave connections in low-rise portal frames. Yee and Melchers [4.48] recognised the difference in connection behaviour where the column was stiffened versus an unstiffened column and provided different values for the four parameters used in prediction equation (4.9) for each case. Only the configuration of four bolts around the tension flange was considered and therefore the model should be used within the bounds of this configuration.

Maquoi and Jaspart [4.51] pointed out that the parameters K_j, K_p and C have to be expressed with the same units as the bending moment capacity M_p in equation (4.9). This is due to the argument of the exponential term being dimensionless, because is, itself, dimensionless. Thus, C wrongly appears to be independent of any unit system as claimed in Table 3 of reference [4.48]. This implies that the values of C should be multiplied by a 10^{-6} and have units of kNm.

As pointed out in reference [4.51], the model takes account of strength and shear deformability of the web of the column. Therefore the rotation, Φ, includes the deformation not only of the connection proper but also of the web of the column.

The predicted curves were found to be in good agreement with test data which consisted of 16 specimens tested by Yee [4.49]. Unfortunately no further comparison has been made due to lack of experimental data for such type of connections. Therefore, before this equation can be used by the designer it should be verified by additional test programs, especially if any of the connection parameters are outside the tested range conducted by Yee [4.49].

4.6 Conclusions

Extended end-plate connections are very popular. They have received the widest study due to the entire fabrication process being possible in the shop under high quality control and can be designed to achieve high connection stiffness and moment capacity, often up to the beam's plastic moment. Data from over 20 separate experimental studies are reviewed with particular reference to the joints in-plane moment-rotation characteristic. More than 150 full-scale tests have been examined and a data bank has been created for such connections. These cover both connections to stiffened and unstiffened columns. Much of the data is for end-plates extended on the tension side only. From the collected experimental data base, the influence of joint parameters on the initial stiffness, moment capacity, failure moment, failure mode of the connection and on the M-Φ behaviour are identified.

Different degrees of moment transfer are expected from these connections and often they do not exhibit the rigid behaviour assumed in a conventional analysis. The available experimental data demonstrated clearly that the response of such connections is nonlinear over almost the entire range of loading. The various mathematical models proposed in the past years, to represent the connection M-Φ behaviour are summarised and reviewed with further modifications.

Extended end-plate connections to stiffened column

Two models are available for extended end-plate connections with column stiffeners. The best approximation of the experimental data was provided by the modified Frye and Morris prediction equation (i.e. equ. (4.10)). The polynomial equation presented by Frye and Morris is empirical and based on a statistical analysis of experimental data. For stiffened connections, the prediction equation does not show any reverse curvature. Despite the fact that the modified Frye and Morris prediction equation does not take account of the preload force of the bolts, material properties of the connected elements and position of the bolts in the end plate (i.e. the gauge and the pitch distances), the proposed equation (4.10) approximates reasonably well the connection behaviour for connections whose failure mode is other than bolt fracture and with a bolt preload force less than 100 kN.

Since predicted curves by the method of Krishnamurthy et al are extremely stiff and are almost straight over the entire range of loading, it is recommended that the equation (4.3) should not be used to predict the moment-rotation relationship for such connections.

Extended end-plate connections to unstiffened column

Again two models are available for extended end-plate connections without column stiffeners. The best approximation of the experimental data was provided by the modified Frye and Morris prediction equation (i.e. equ. (4.11)). The polynominal curves approximates reasonably well the experimental curves up to rotation of about 0.02 radians after which deviation is substantial for connections whose ratio of column flange thickness to beam depth is greater or equal to 0.045.

Tarpy and Cardinal's prediction equation is based on an elastic computer analysis. The predicted curve is almost a straight line and deviates rapidly from the true connection behaviour. Even at very low moment, the prediction equation does not approximate closely the initial connection tangent stiffness. It is strongly recommended not to use this equation for predicting the moment-rotation relationship. The predicted moment connection capacity is in considerable error in comparison to the experimental result.

Yee and Melchers prediction equation is a nonlinear mathematical model suitable for bolted end-plate connections that are used for eaves connections in low-rise portal frames. This physically based approach to the prediction of moment-rotation curves takes into account the possible failure modes and the deformation characteristics of the connection elements. The method used is promising with alterations of the model to extent to represent the behaviour of beam-to-column connections in the lower storeys of high-rise frames when the axial load effect in the column is significant would result is an acceptable equation.

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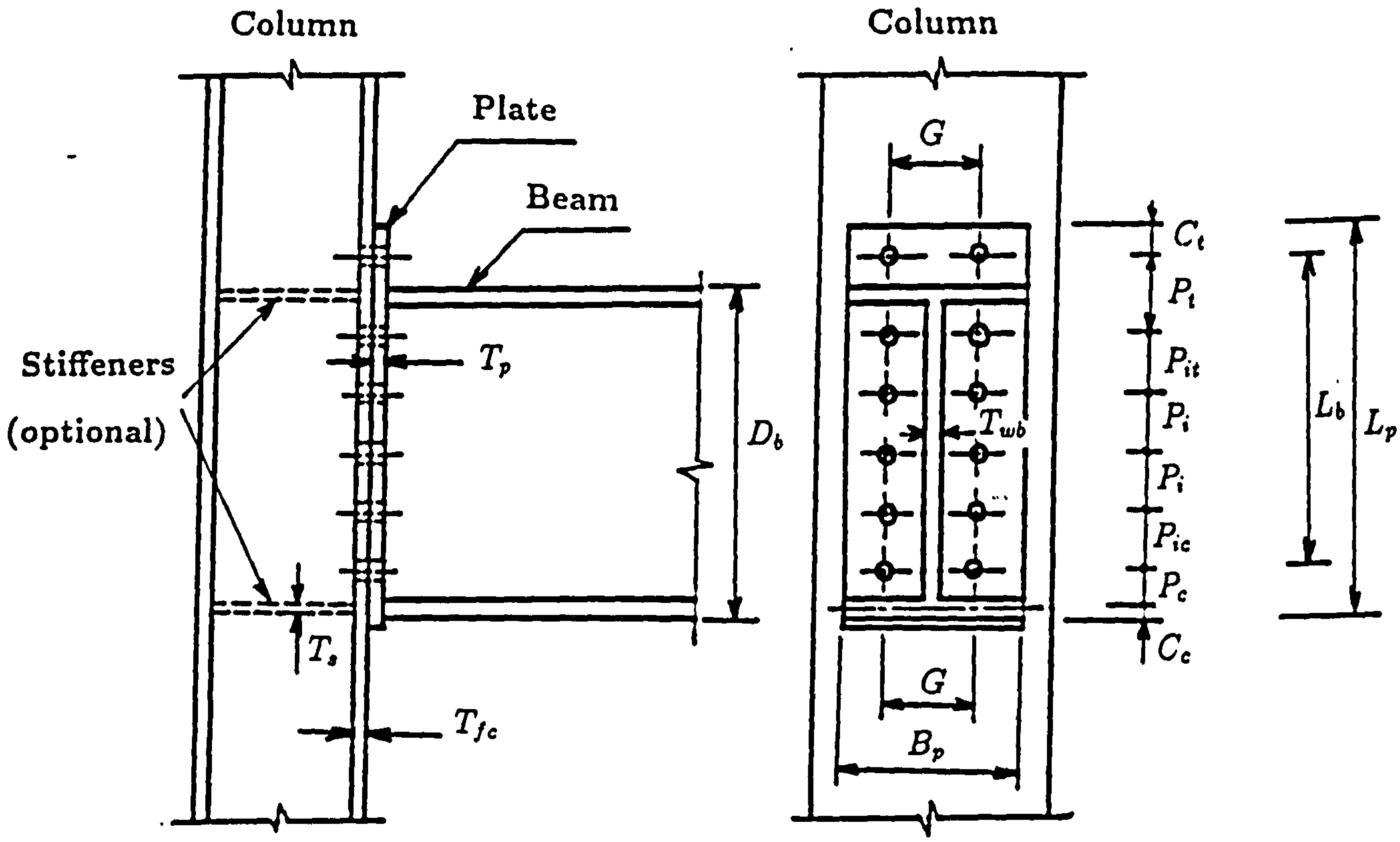
Table 4.1 Available experimental data for extended end-plate connections

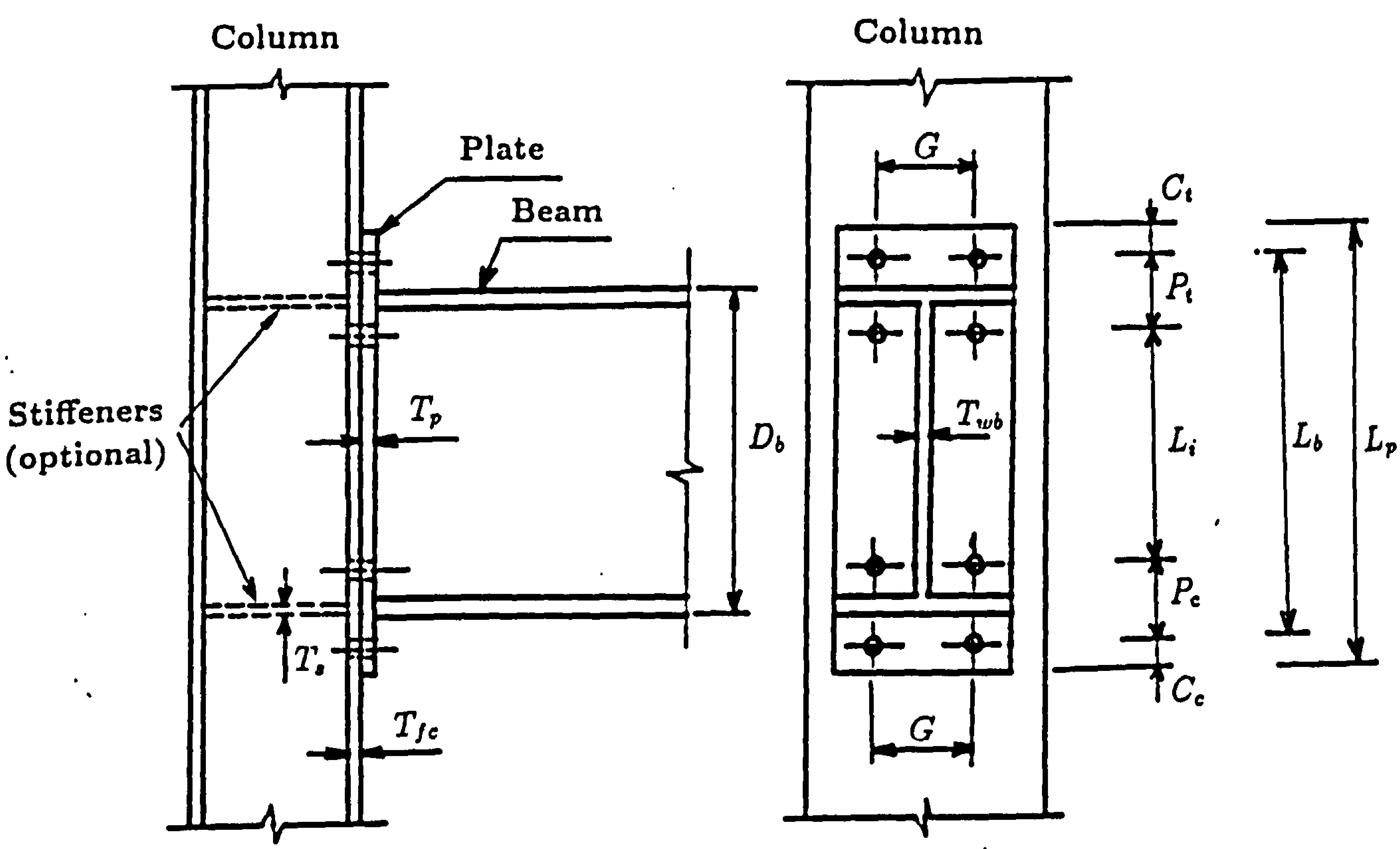
Reference of experimental data	Date	Country of origin	Number of tests	Type of fastener	Test type	Comments on M-Φ curves
Johnson et al [4.22]	1960	U.K.	1 stiffened	Cruciform	¾ in HT bolts	Provided for each test.
Sherbourne [4.10]	1961	U.K.	1 unstiffened 4 stiffened	Cruciform	¾ & ⅞ in HT bolts	Provided for each test.
Mann [4.25]	1968	U.K.	5 unstiffened stiffened	Cruciform	¾, 1 & 1.1/8 in HSFG bolts	Provided for each in test.
Bailey [4.26]	1970	U.K.	3 unstiffened 10 stiffened	Cruciform	HSFG bolts	Provided for each test.
Zoetemeijer [4.12]	1974	Netherland	8 unstiffened	4 Cantilever 4 Portal frame	M20 & M22 Gr 10.9 bolts	Provided for 4 tests only.
Packer [4.3]	1977	U.K.	4 ustiffened 1 stiffened column	Cruciform load in	M16 HSFG bolts	Provided for each test.
Ioannides [4.27]	1978	U.S.A.	6 unstiffened	Cruciform	¾, ⅞ & 1 in A325 bolts	Provided for each test.
Dews [4.28]	1979	U.S.A.	3 unstiffened	Cruciform	¾ & 1 in A325 bolts	Provided for each test
Grundy et al [4.11]	1980	Australia	2 unstiffened	Cruciform load in column	⅞ HT bolts	Provided for each test.

Table 4.1 continued

Johnstone & Walpole [4.29]	1981	New Zealand	8 unstiffened	-	M30 & M24 Gr. 8.8 bolts	Provided for each test.
Tarpy & Cardinal [4.21]	1981	U.S.A.	14 unstiffened 2 stiffened	Cruciform load in column	$\frac{3}{4}$, $\frac{7}{8}$ & 1 in A325 bolts	Provided for 4 tests.
Graham [4.31]	1981	U.K.	21 unstiffened	Cruciform load in column	M16 HSFG bolts	Provided for each test.
Moore & Sims [4.35]	1983	U.K.	2 unstiffened 2 stiffened	Cruciform	M16 Gr 8.8 bolts	Provided for two only.
Zoetemiejer & Munter [4.36]	1984	Netherland	4 unstiffened	Cruciform	M20 Gr 8.8 bolts	Provided for each test.
Tong [4.37]	1985	U.K.	1 unstiffened 11 stiffened	Cruciform	M20 Gr 8.8 bolts	Provided for each test.
Zandonini & Zanon [4.2, 4.40]	1986	Italy	10 stiffened	Cantilever	M20 Gr 8.8 bolts	Provided for each test.
Prescott [4.1]	1987	U.K.	4 stiffened	Cruciform	M20 Gr 8.8 bolts	Provided for each test.
Davison [4.41]	1987	U.K.	4 stiffened	Cruciform load in column	M16 Gr 8.8 & HSFG	Provided for each test.
Chakrabarti [4.42]	1987	U.K.	1 unstiffened	Cruciform	M16 Gr 8.8 bolts	Provided for each test.
Janss et al [4.43]	1987	Belgium	6 unstiffened	Cantilever	Grade 10.9 H.S. bolts	Provided for each test.



(a) Extended end-plate on tension side only.



(b) Extended end-plate on tension and compression side.

Figure 4.1 Typical extended end-plate connection.

FIG.4-2 Comparison of moment-rotation curves for a flush and extended and plate connection. (a)

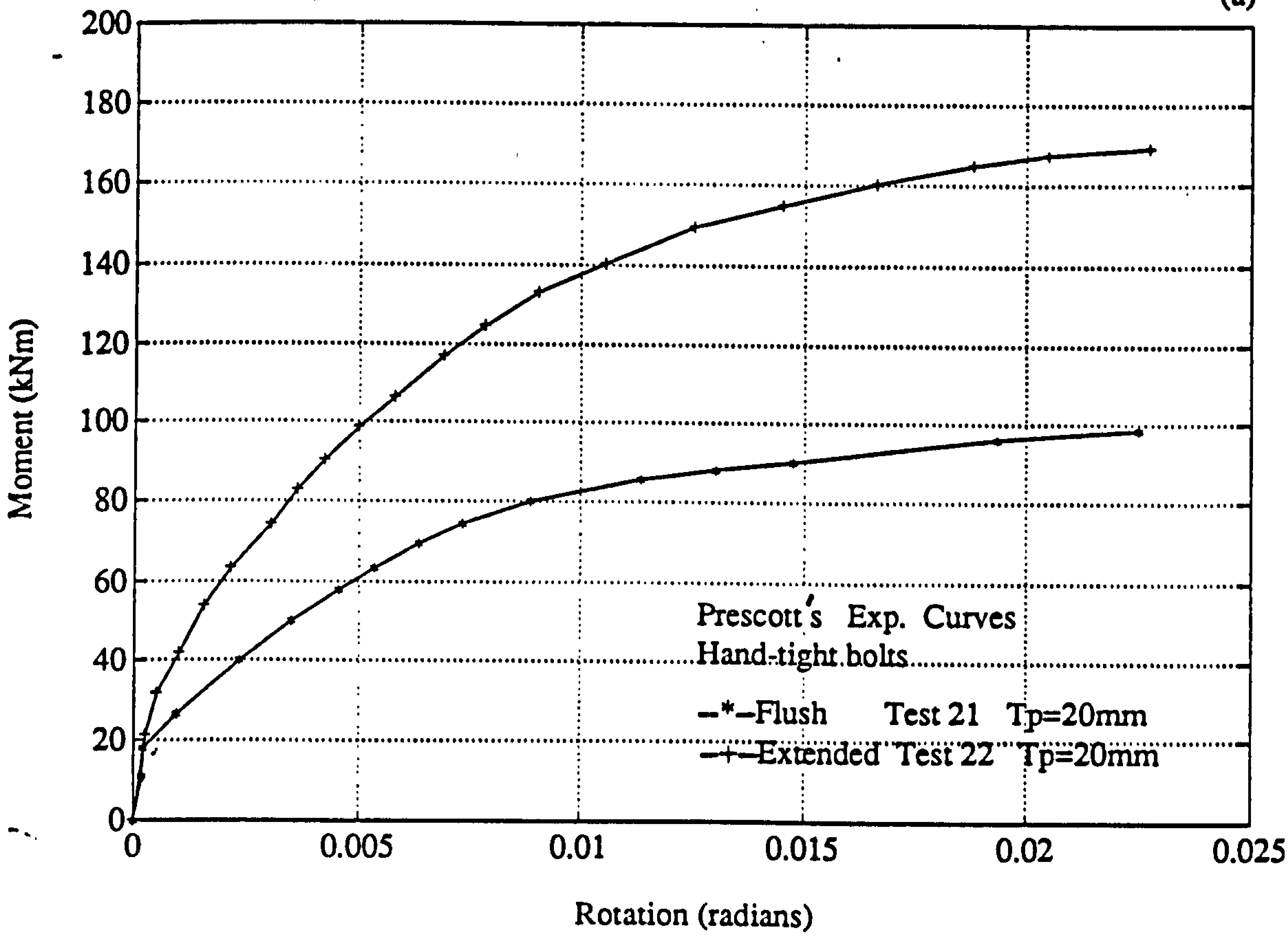
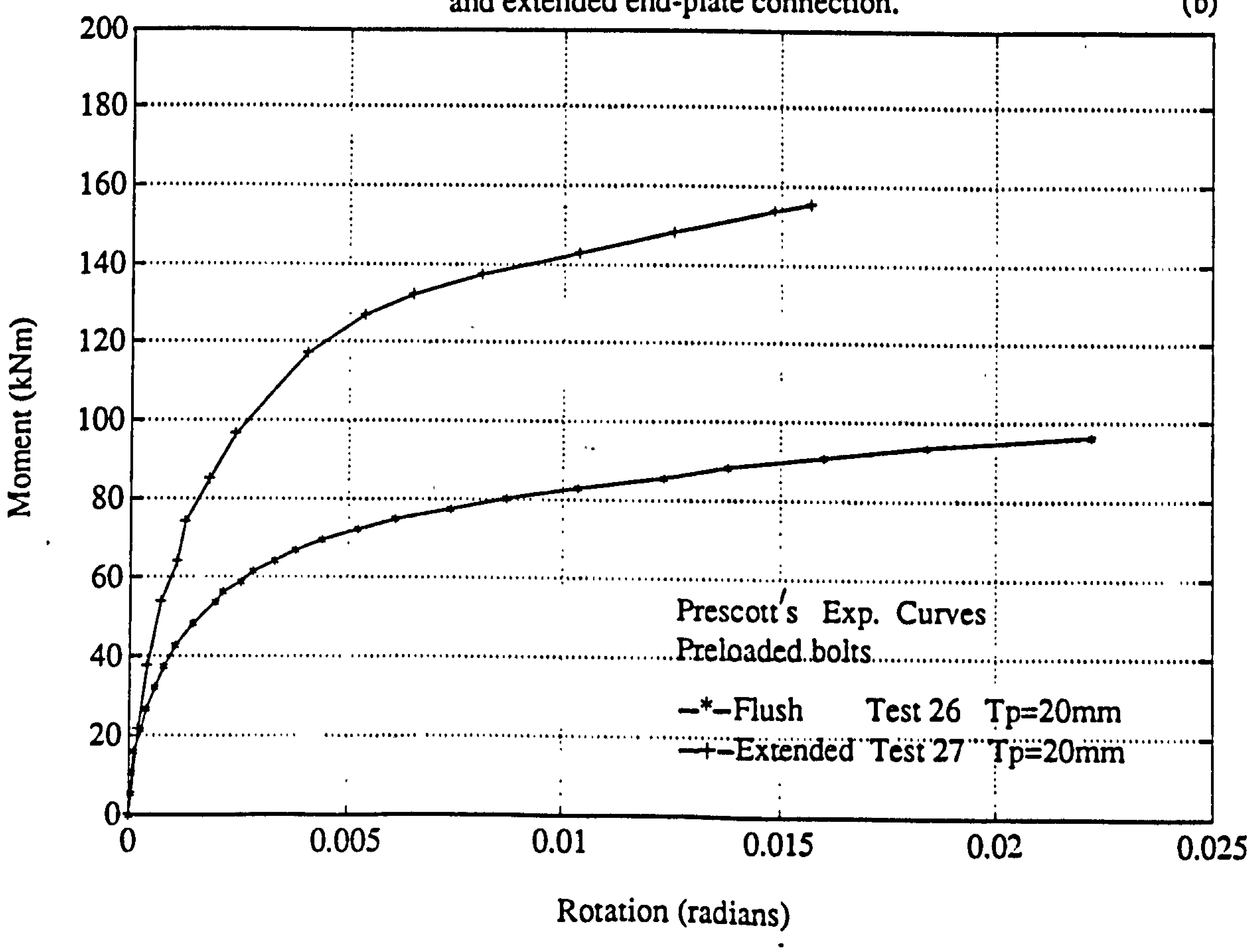


FIG.4-2 Comparison of moment-rotation curves for a flush and extended end-plate connection. (b)



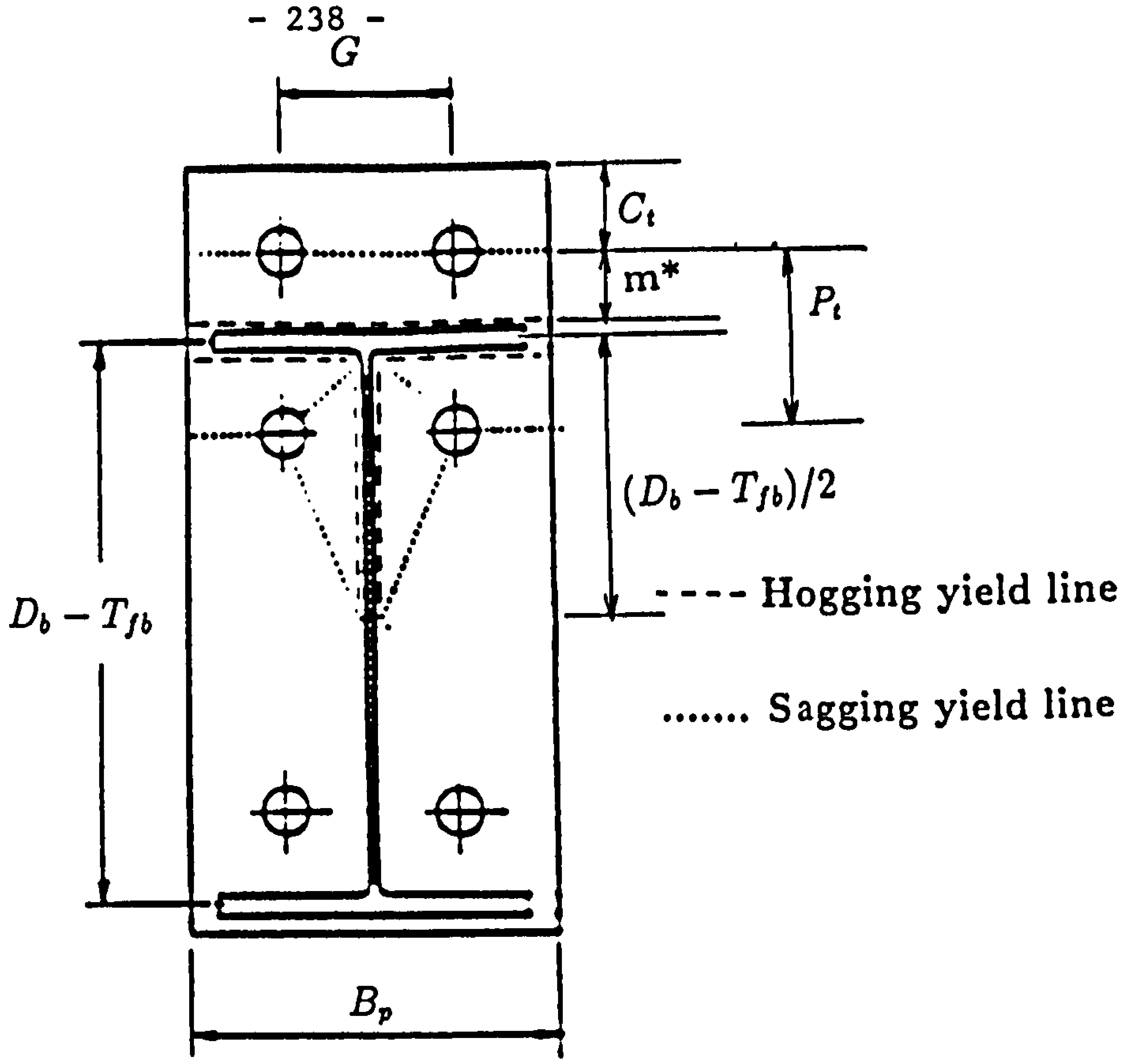


Figure 4.2c Yield line pattern for end-plate [4.16].

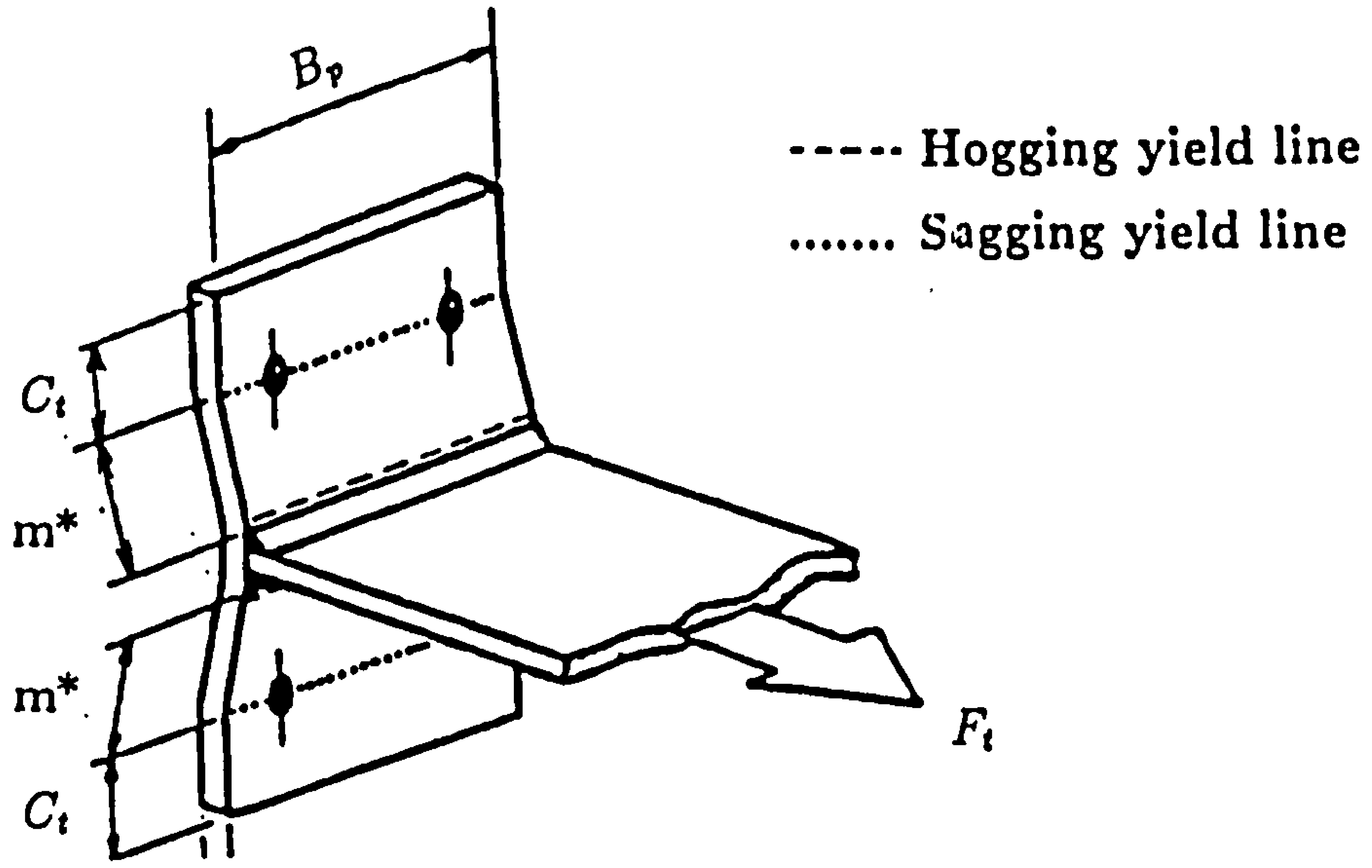


Figure 4.2d Equivalent T-stub model for end-plate [4.16].

FIG.4-3 Effect of end-plate thickness
on moment-rotation characteristics

(a)

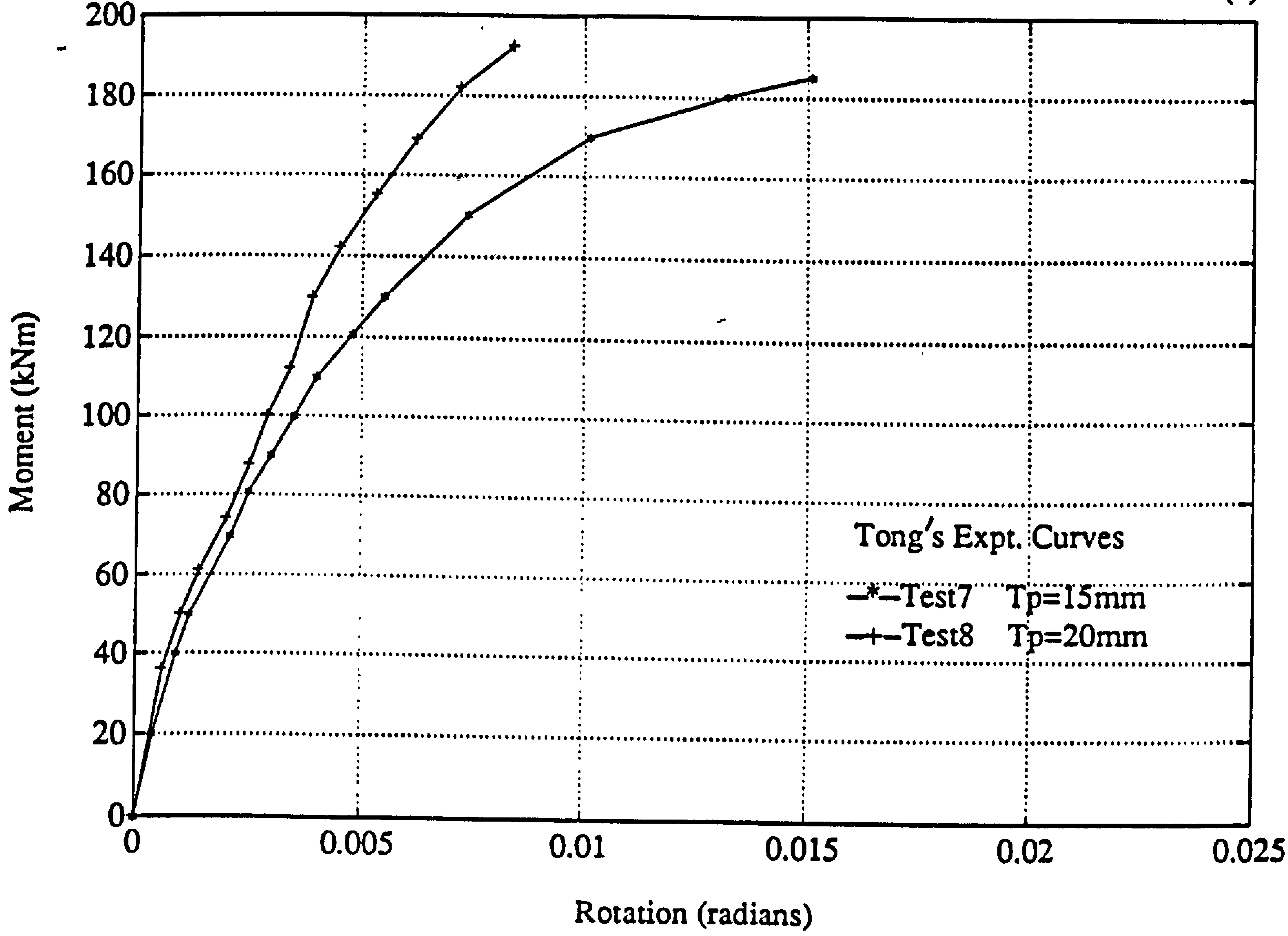


FIG.4-3 Effect of end-plate thickness
on moment-rotation characteristics

(b)

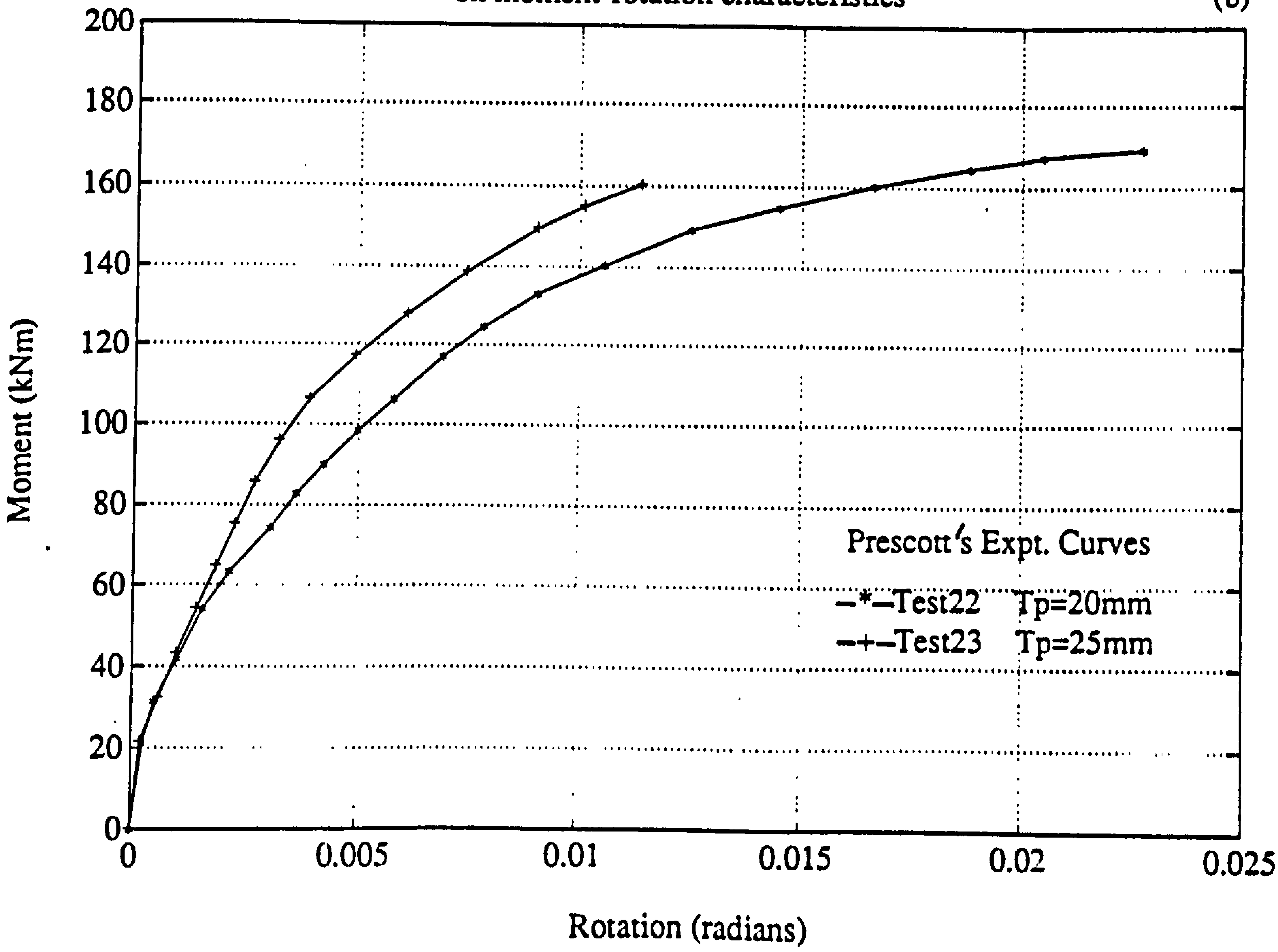


FIG.4-4 Effect of end-plate depth on moment-rotation characteristics.

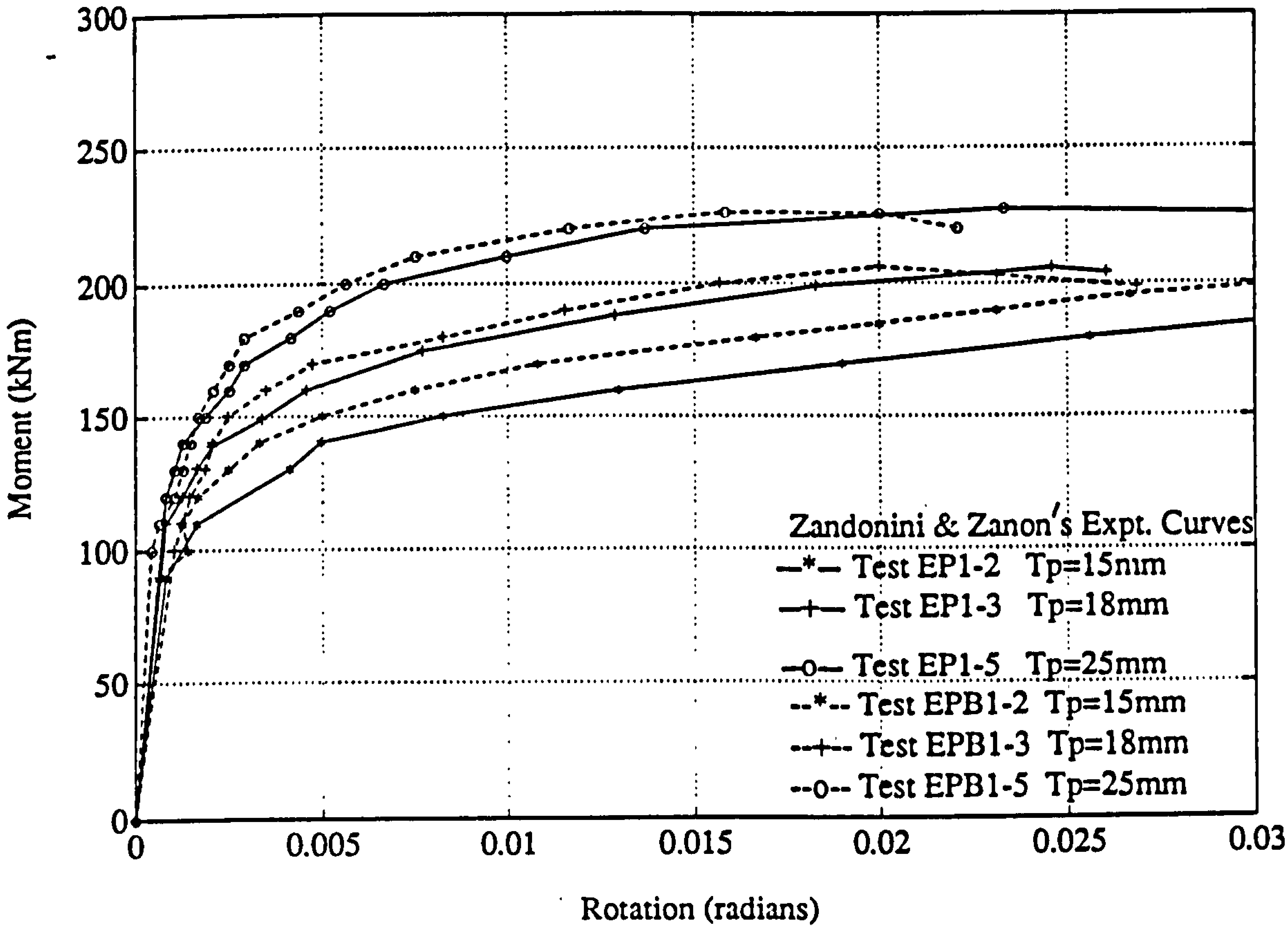


FIG.4-5 Effect of gauge distance on moment-rotation characteristics.

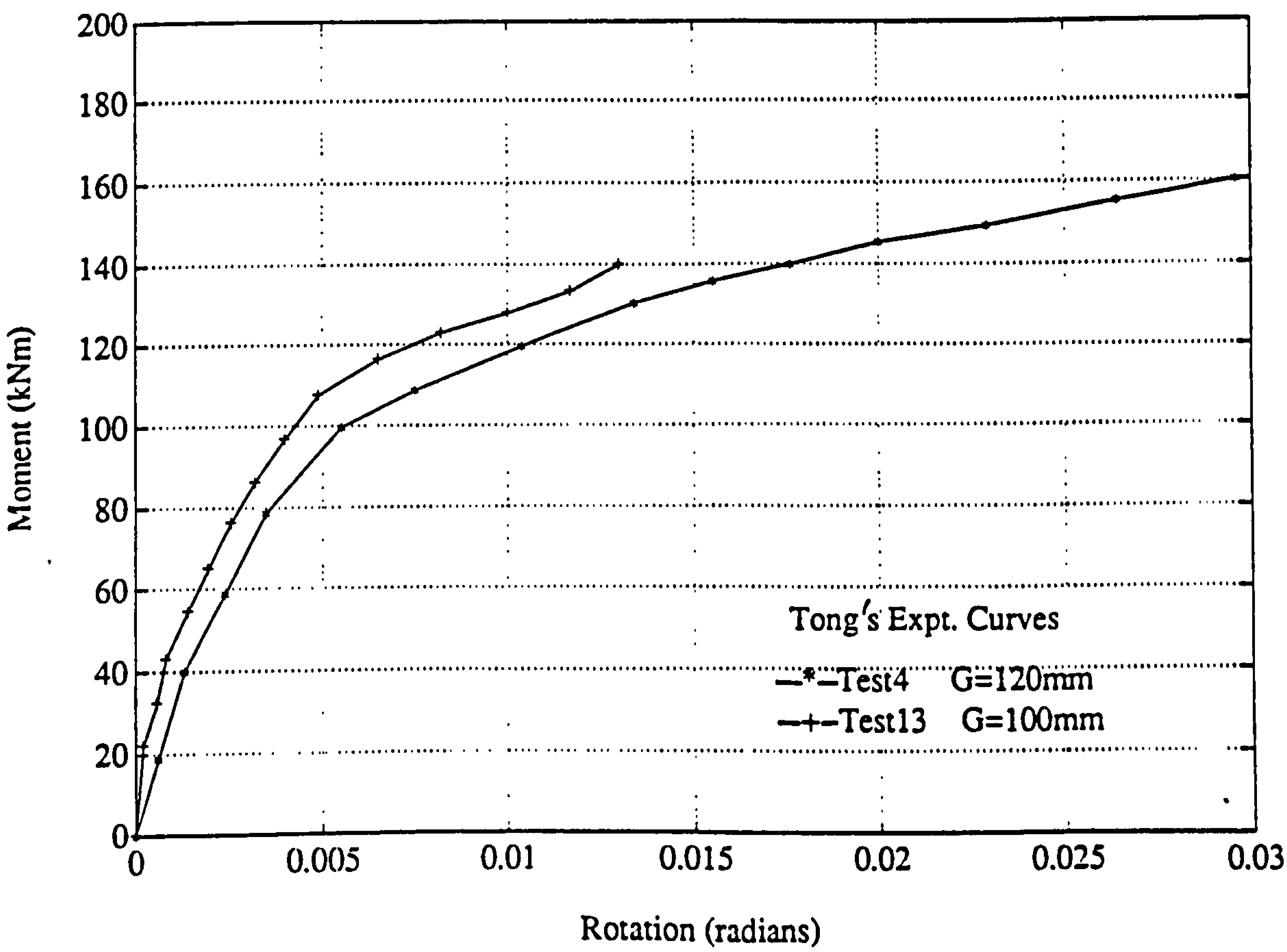


FIG.4-6 Effect of bolt preload on moment-rotation characteristics.

(a)

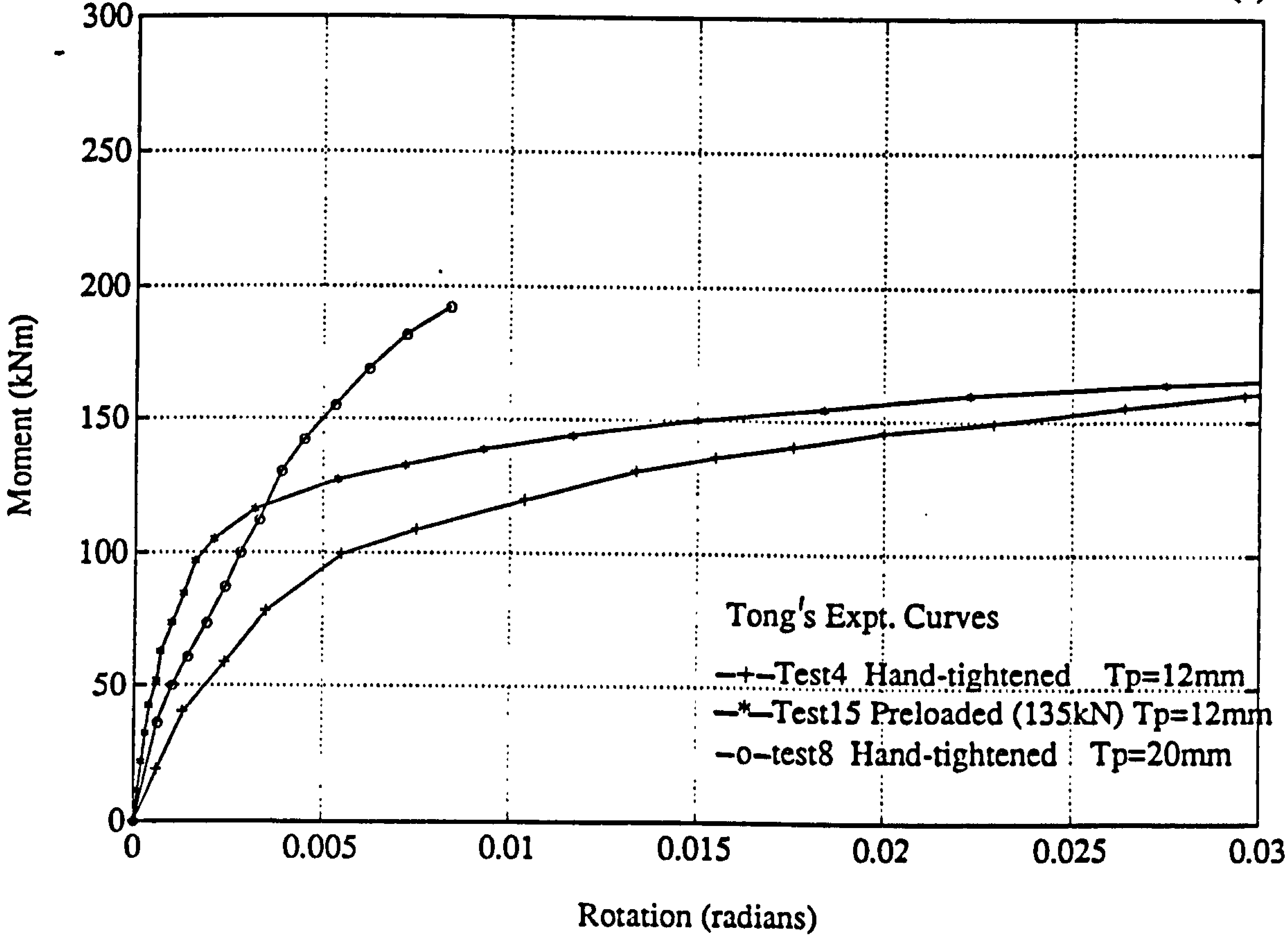


FIG.4-6 Effect of bolt preload on moment-rotation characteristics.

(b)

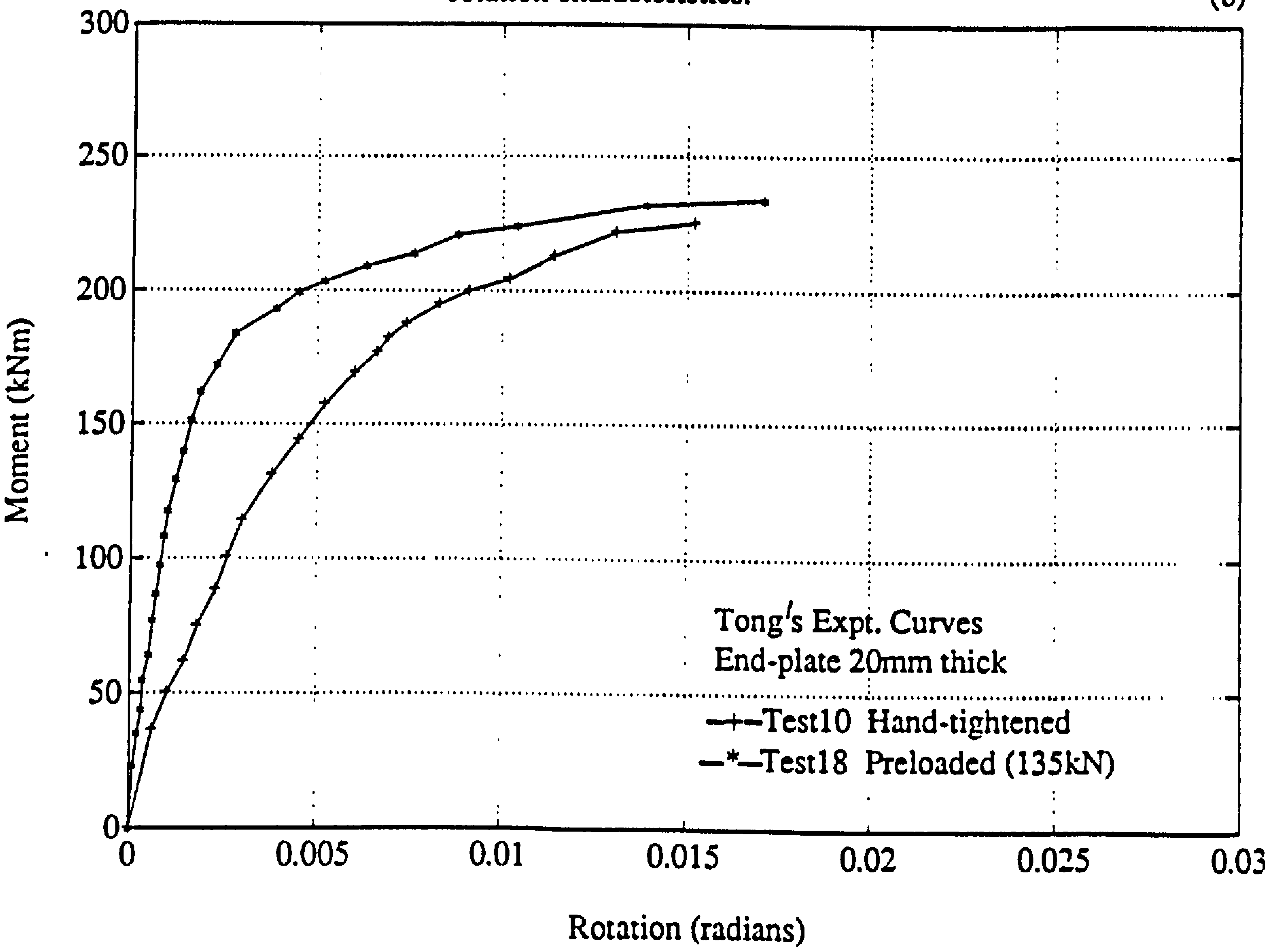


FIG.4-6 Effect of bolt preload on moment-rotation characteristics.

(c)

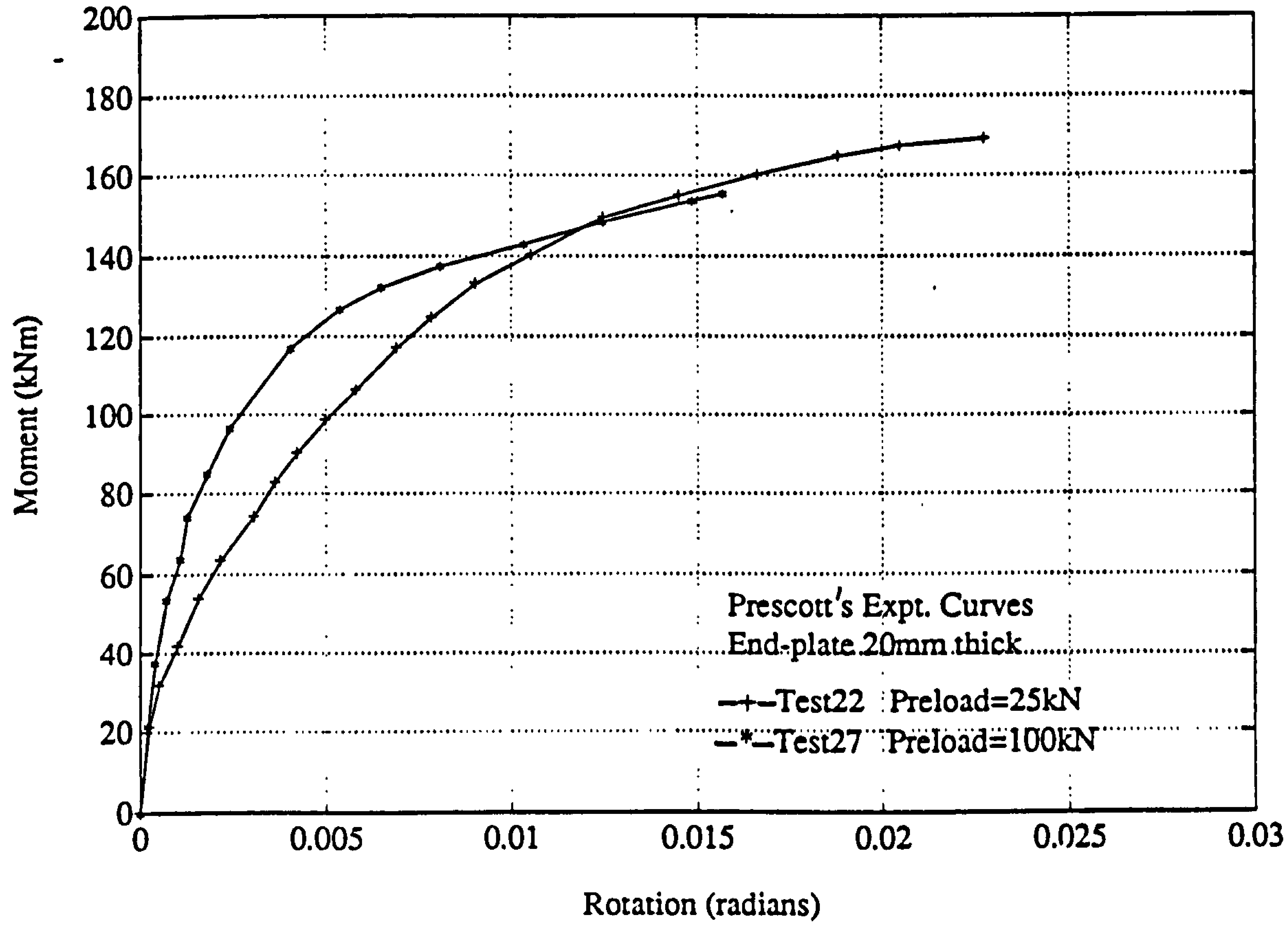


FIG.4-7 Effect of column section on moment-rotation characteristics.

(a)

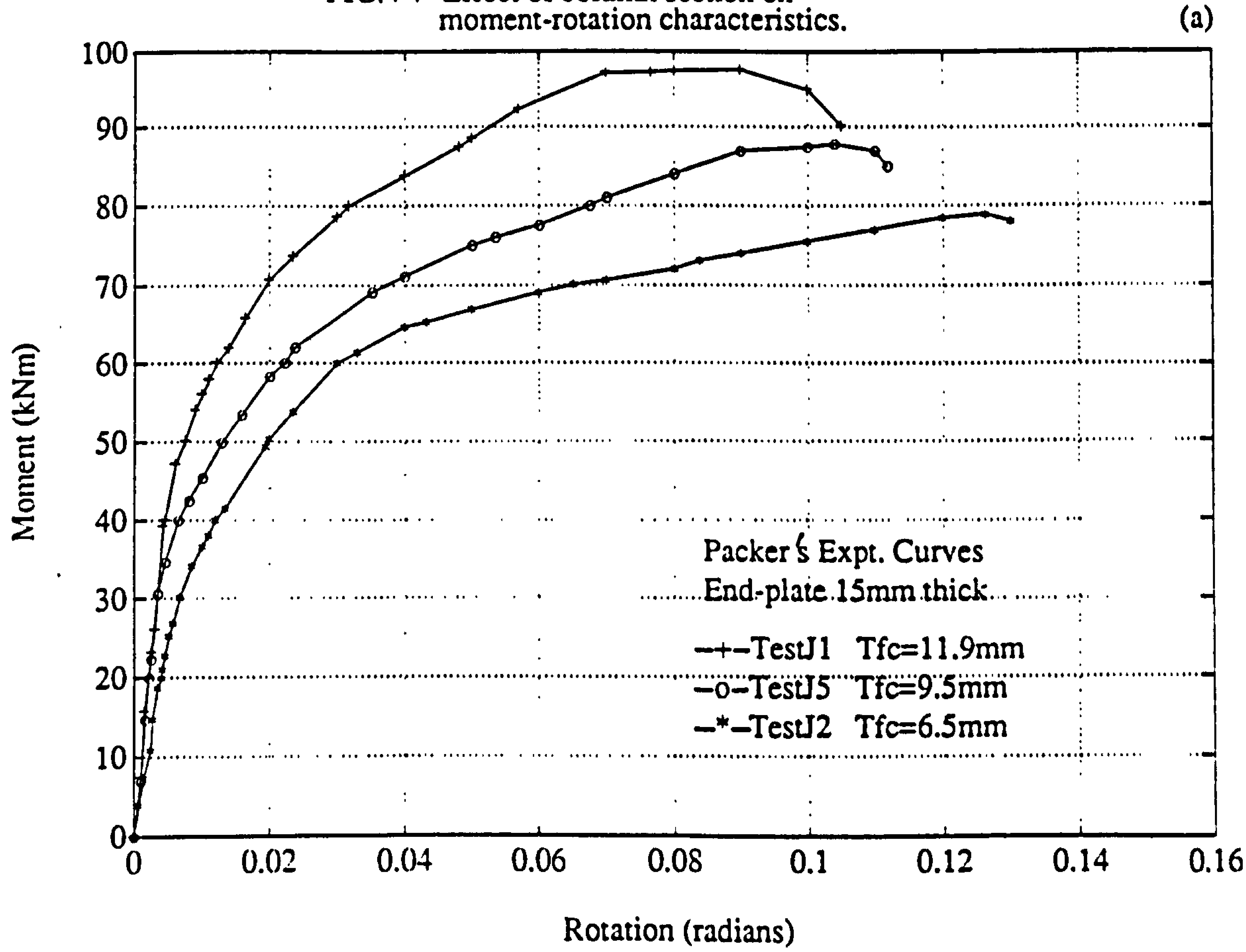


FIG.4-7 Effect of column section on moment-rotation characteristics.

(b)

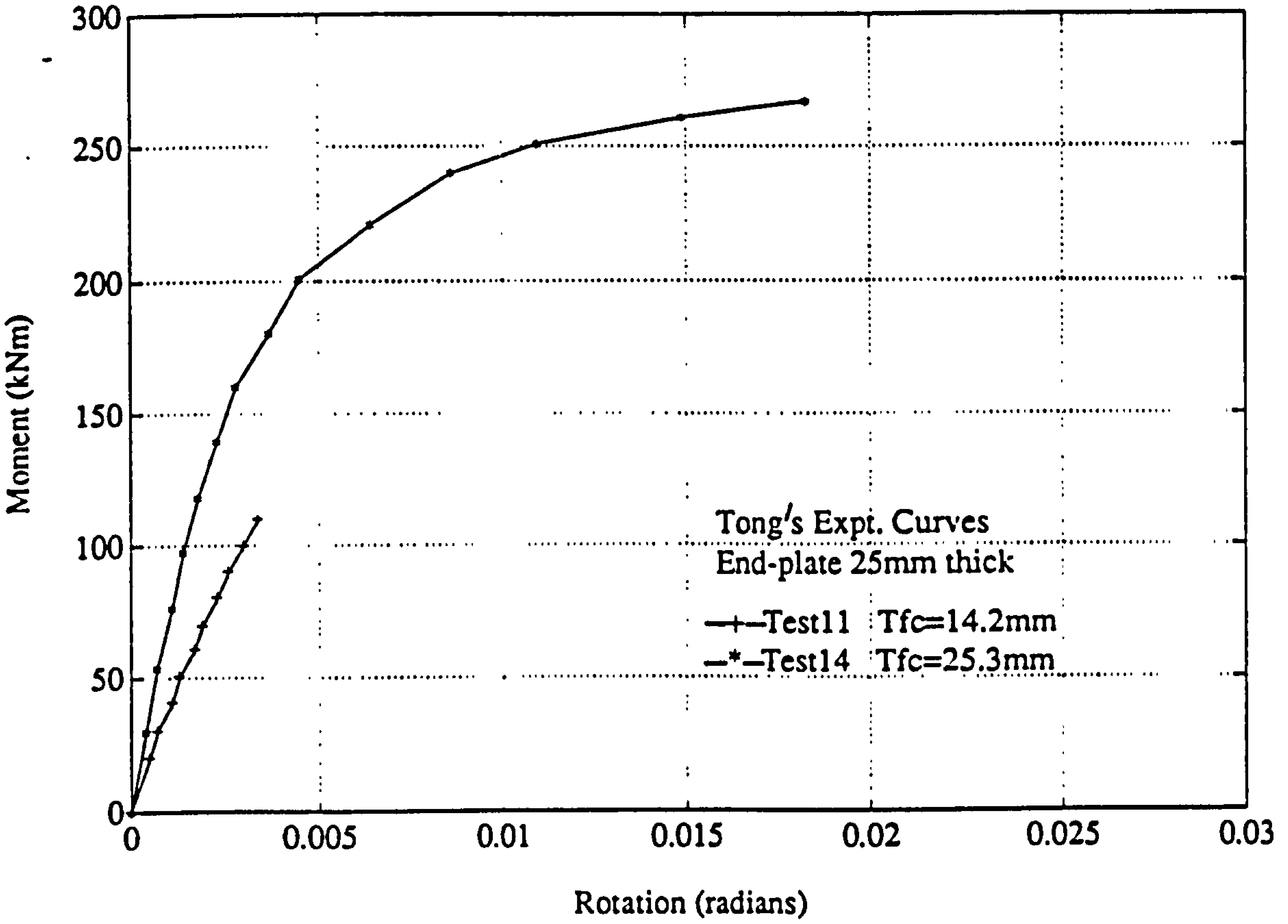


FIG.4-8 Effect of column web stiffeners on moment-rotation characteristics.

(a)

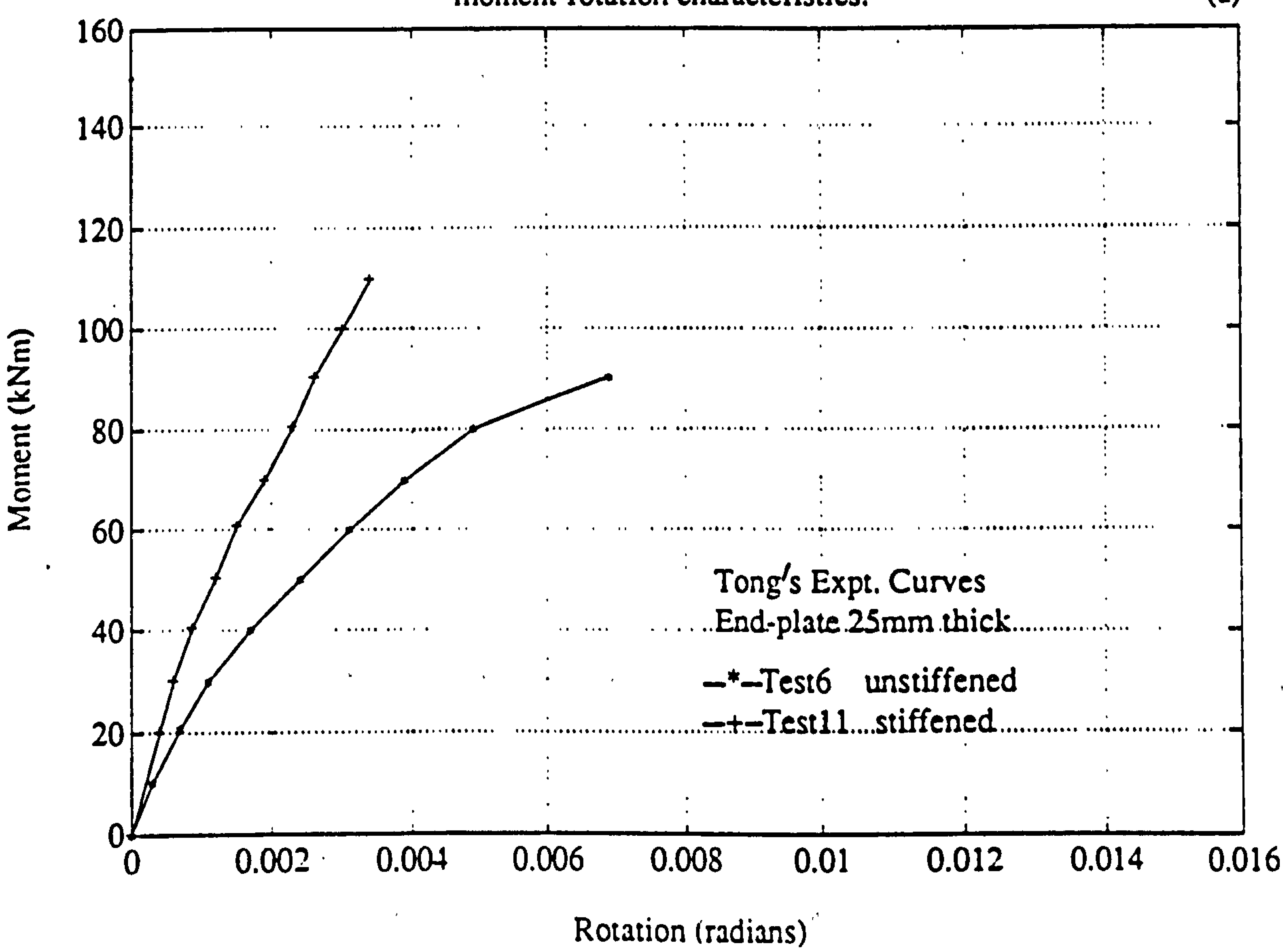
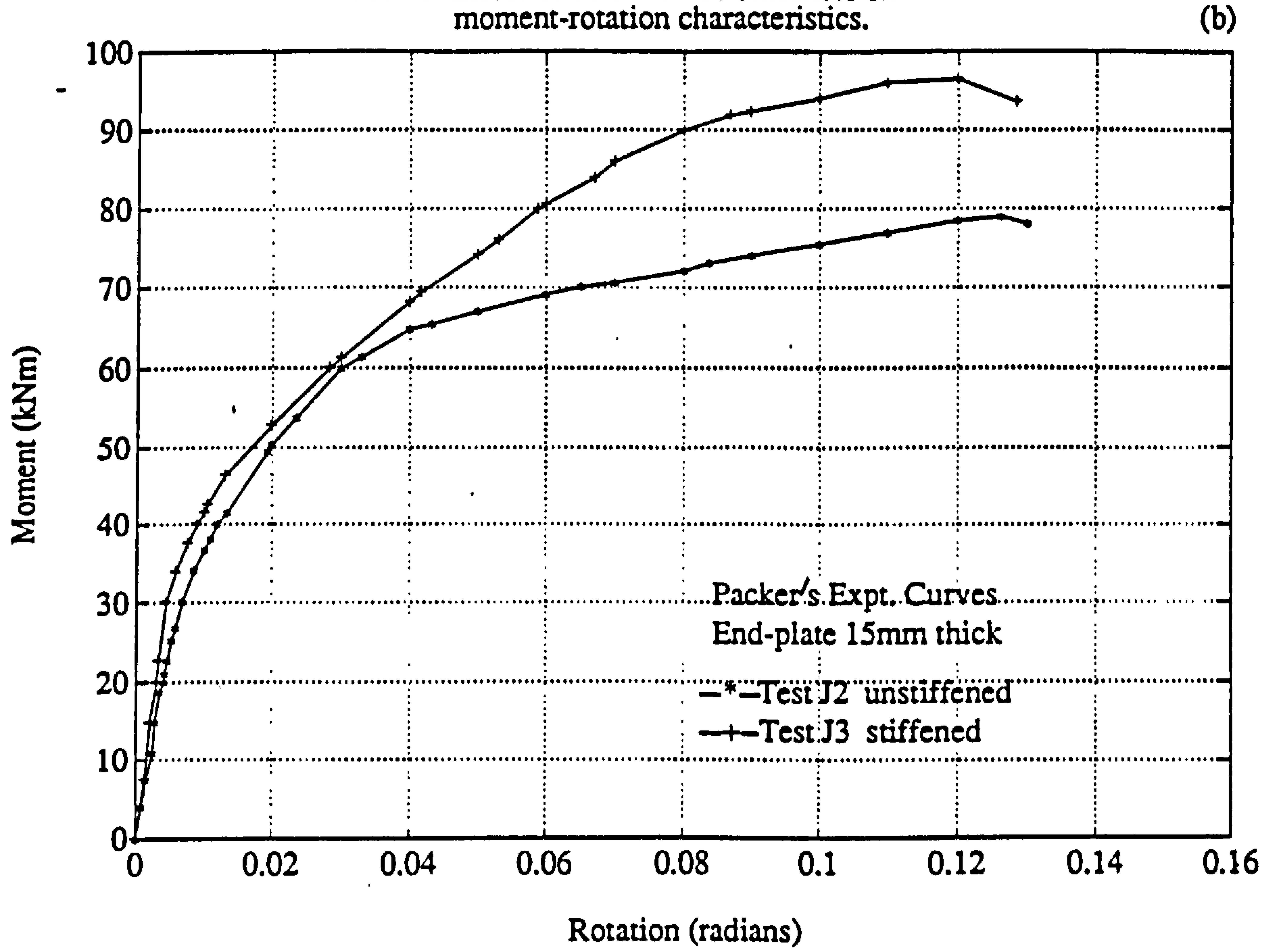


FIG.4-8 Effect of column web stiffeners on moment-rotation characteristics.



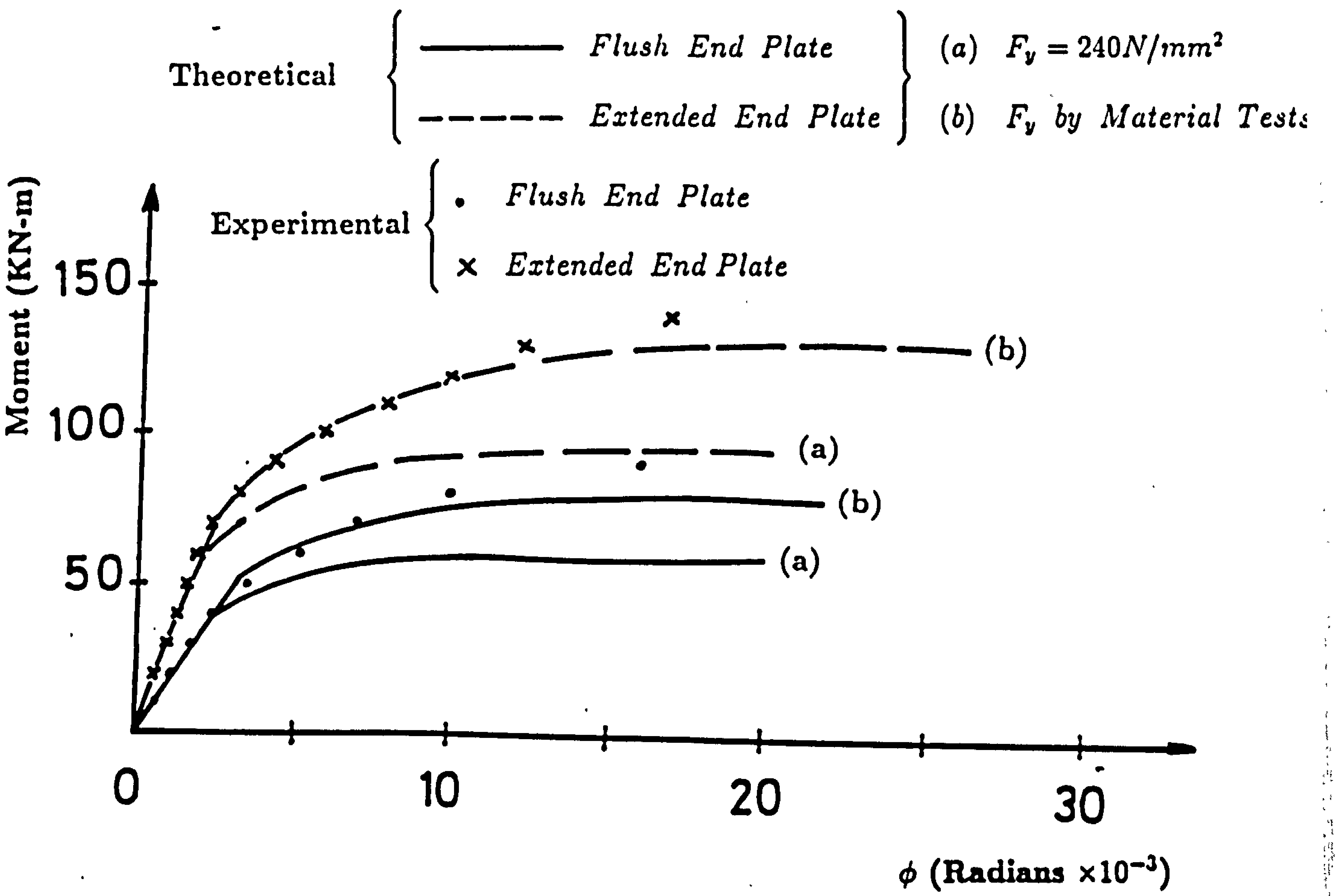


Figure 4.9 Effect of actual yield stress on moment-rotation characteristics [4.1].

FIG.4-10 Effect of shear on moment-rotation characteristics.

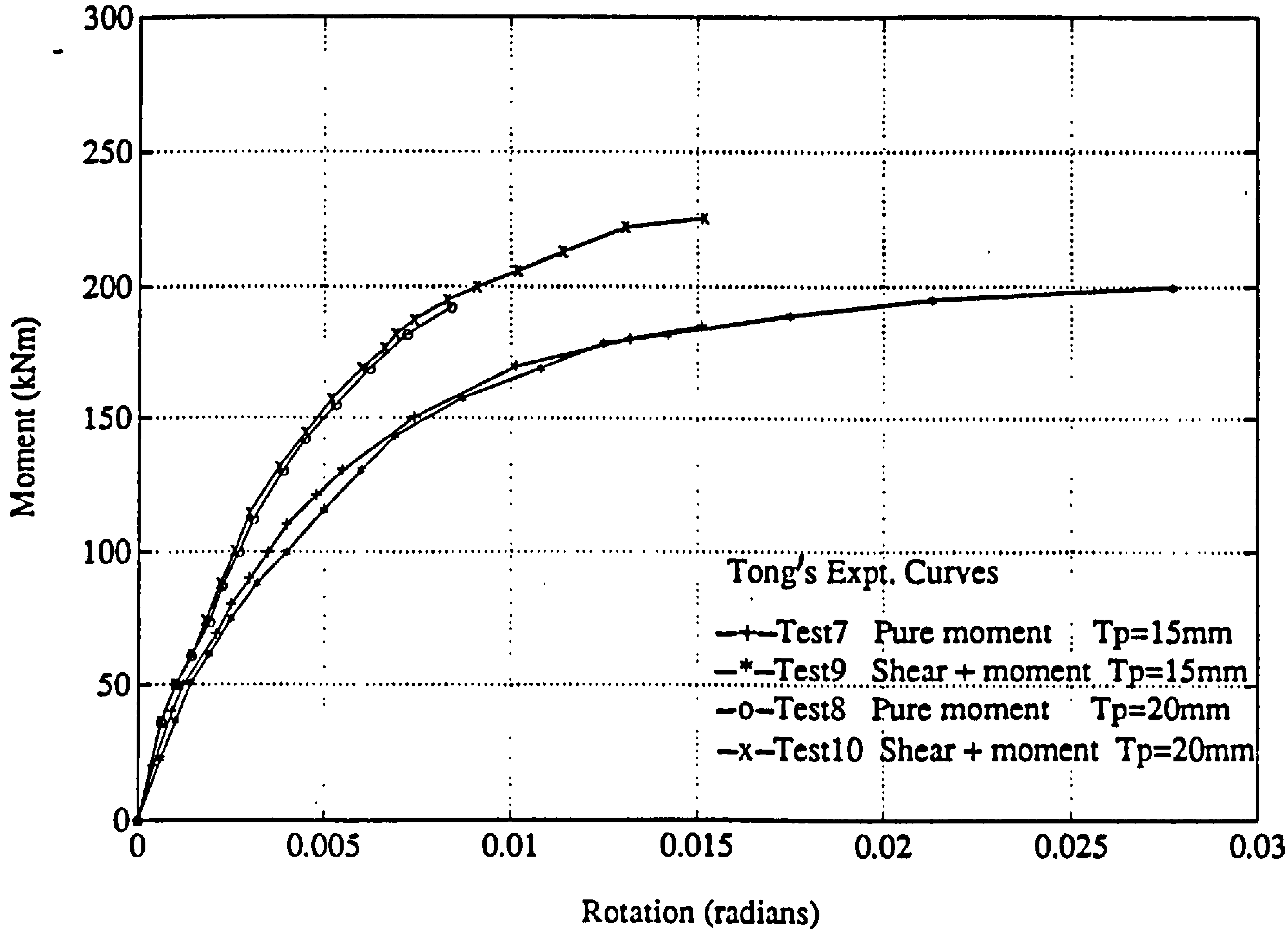


FIG.4-11 Effect of axial load on moment-rotation characteristics.

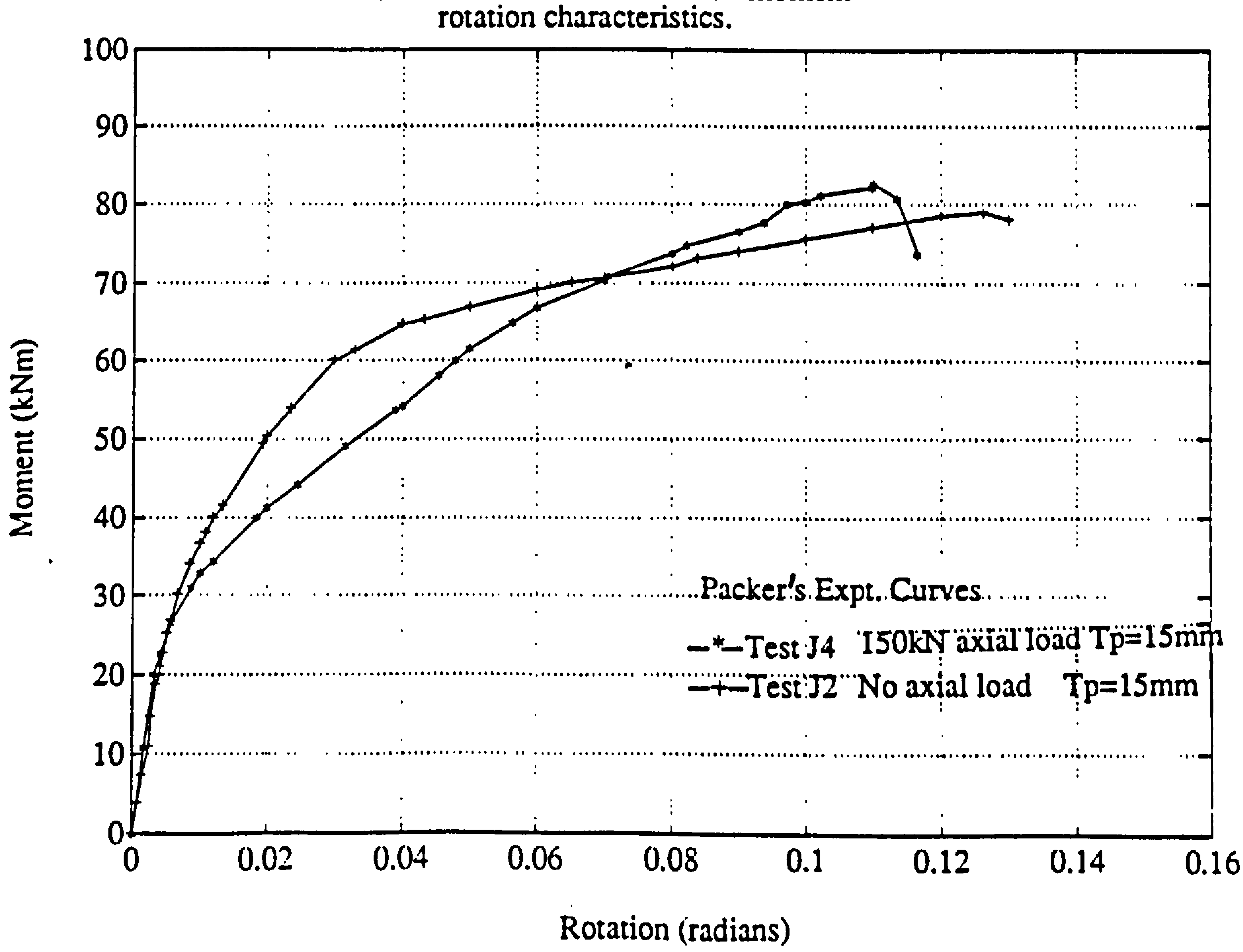


FIG.4-12 Comparison between analytical moment-rotation relationships and Johnson et al. test5 (stiffened column)

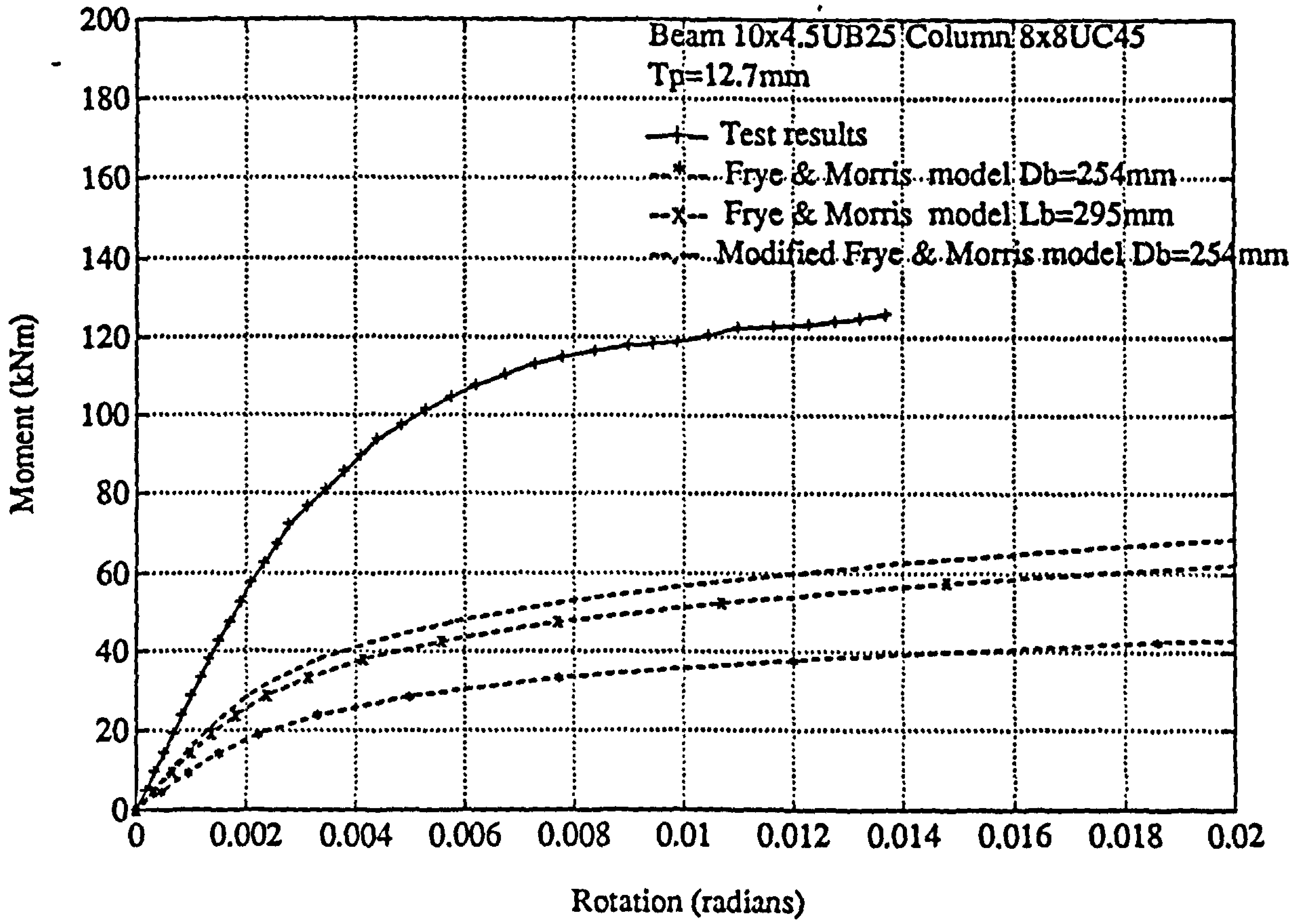


FIG.4-13 Comparison between analytical moment-rotation relationships and Sherbourne test A2 (stiffened column)

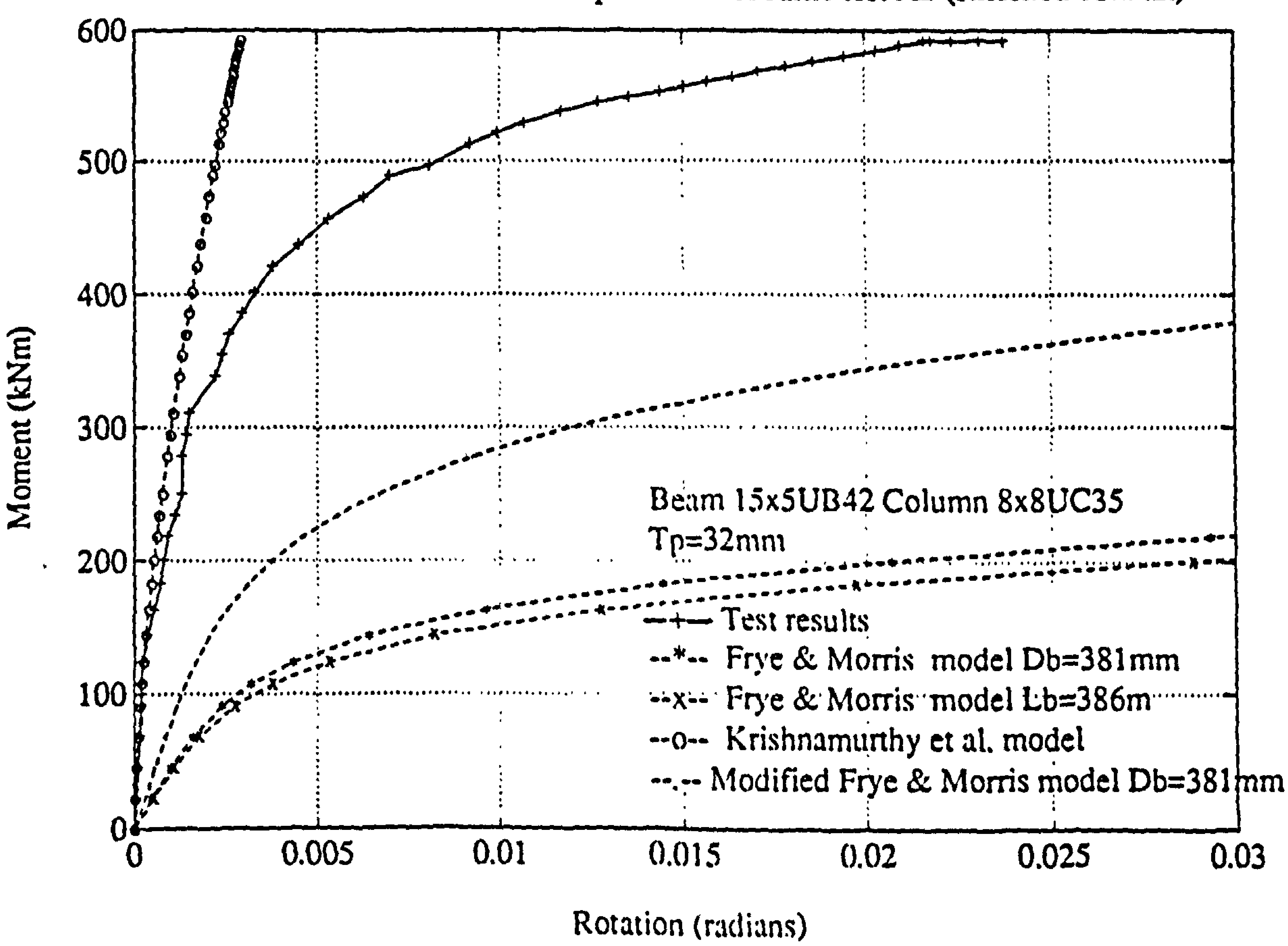


FIG.4-14 Comparison between analytical moment-rotation relationships and Sherbourne test A3 (stiffened column)

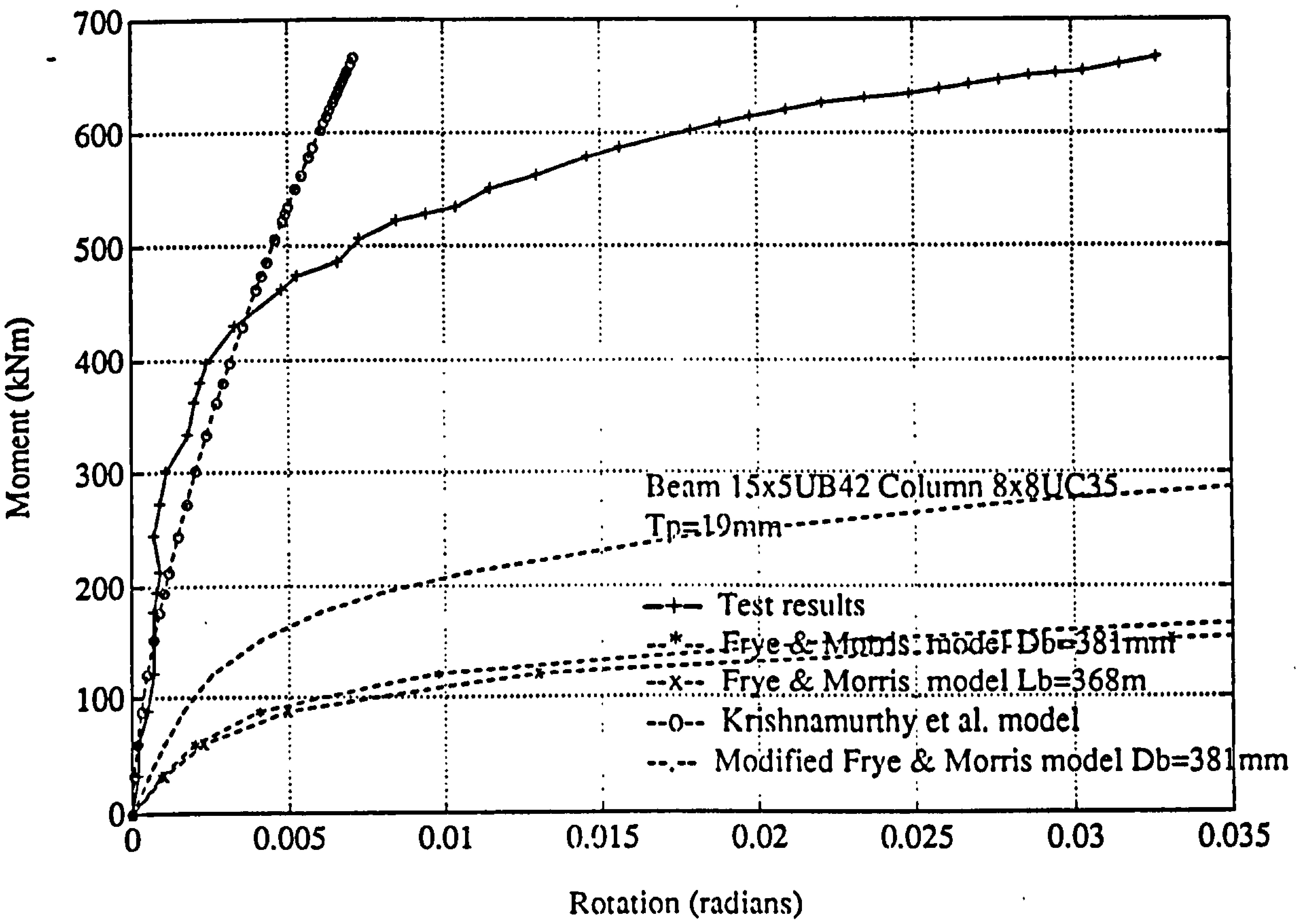


FIG.4-15 Comparison between analytical moment-rotation relationships and Sherbourne test B1 (stiffened column).

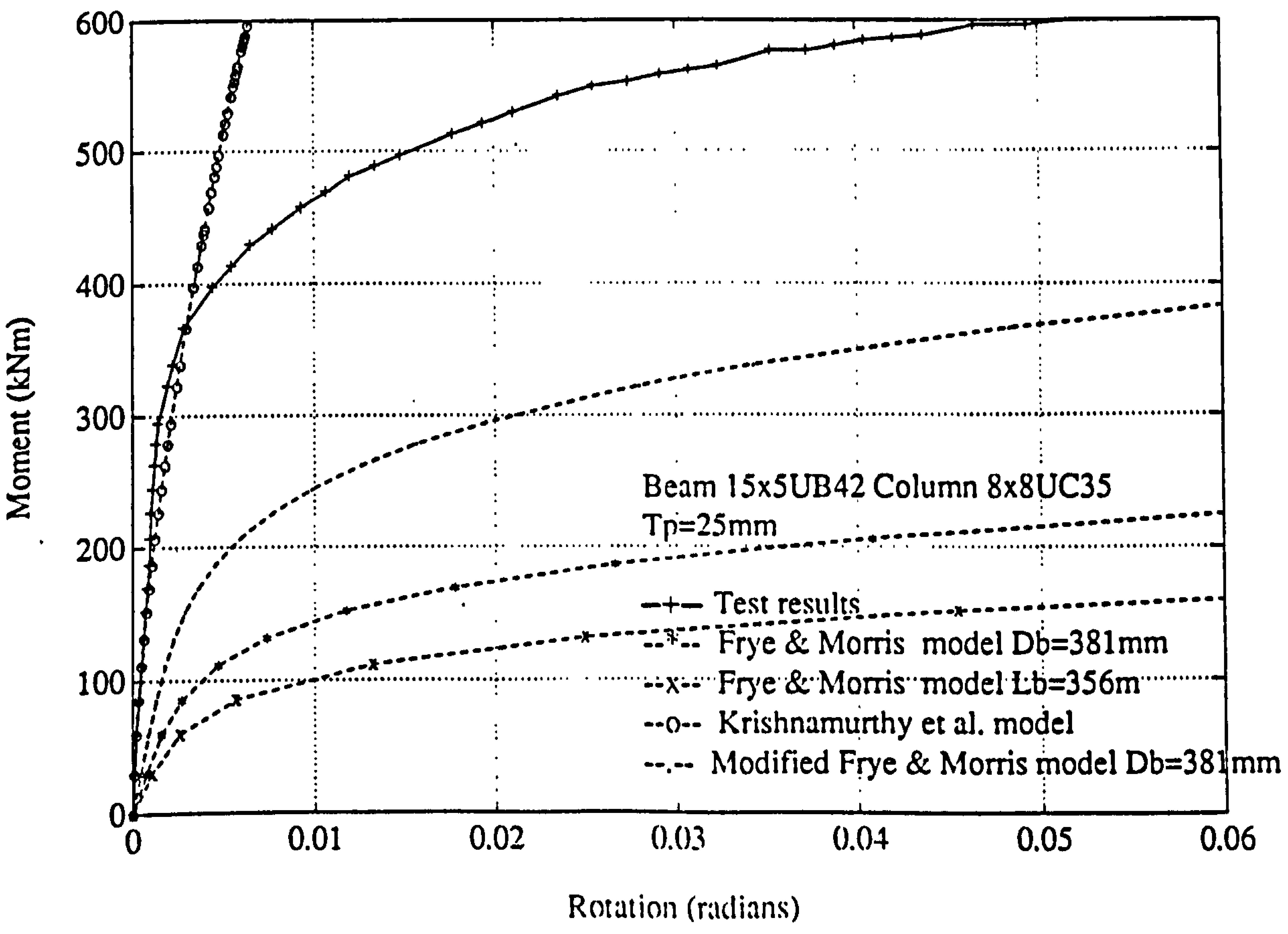


FIG.4-16 Comparison between analytical moment-rotation relationships and Sherbourne test B2 (stiffened column).

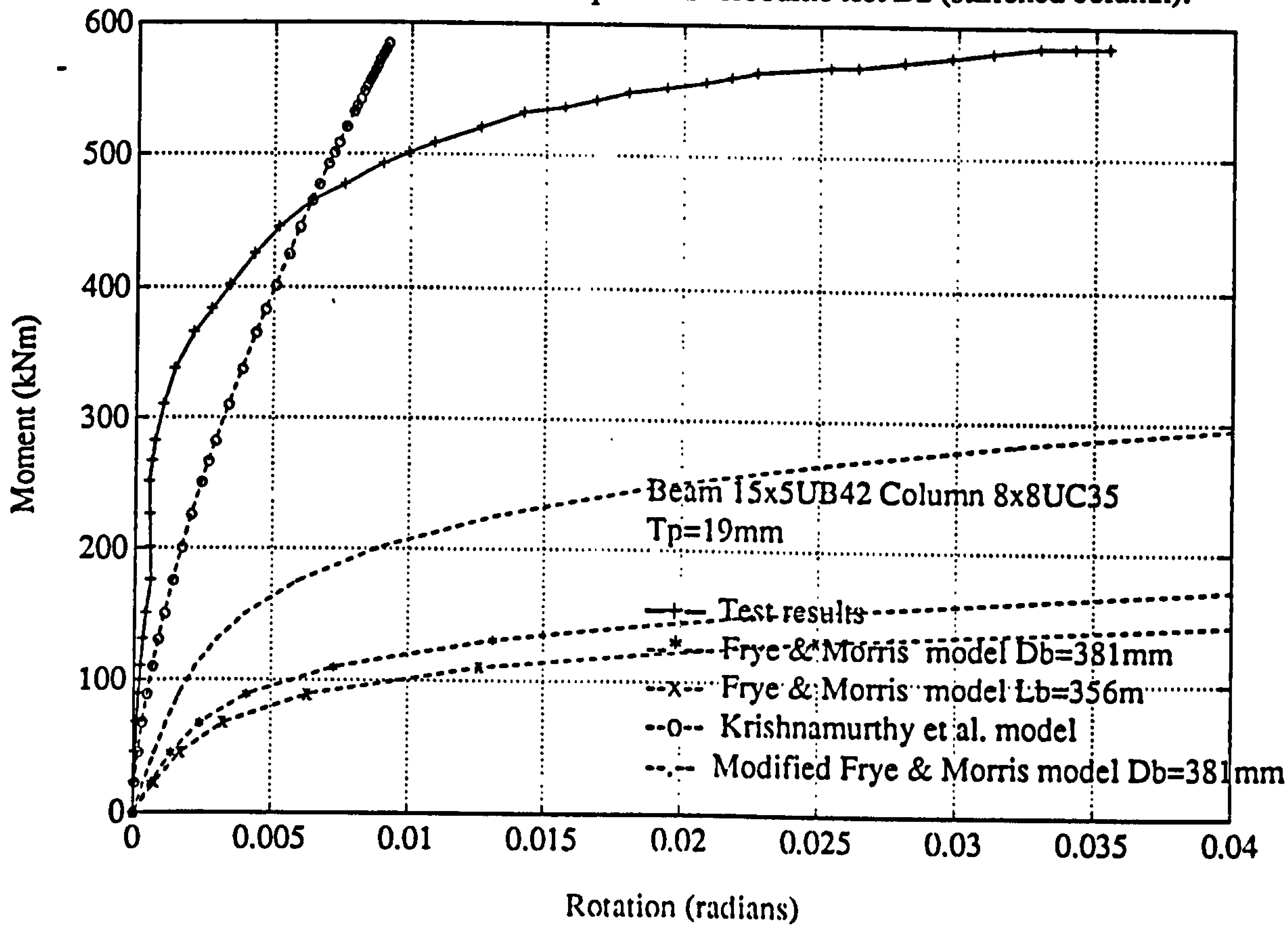


FIG.4-17 Comparison between analytical moment-rotation relationships and Mann test C6 (stiffened column)

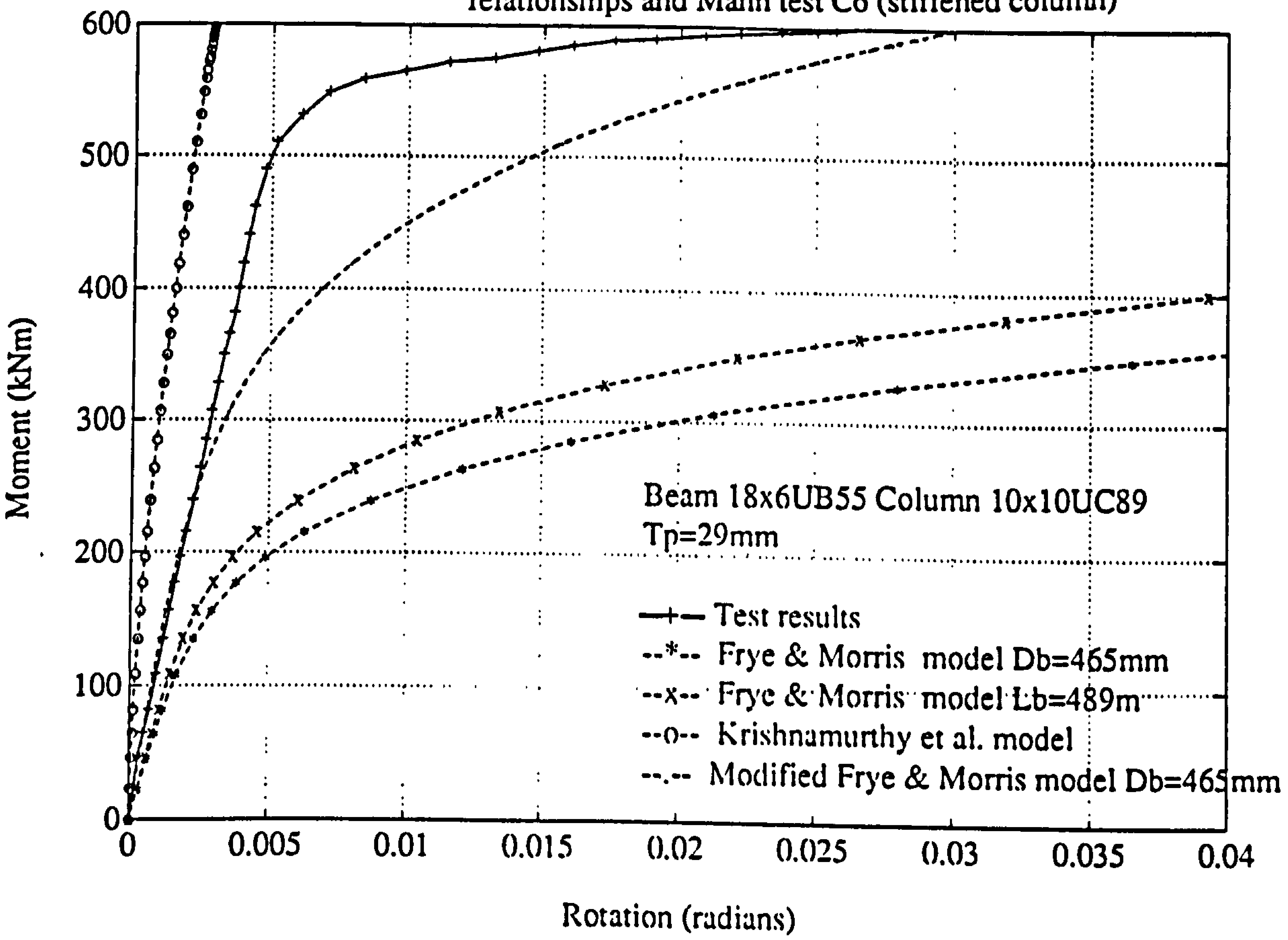


FIG.4-18 Comparison between analytical moment-rotation relationships and Packer test J3 (stiffened column)

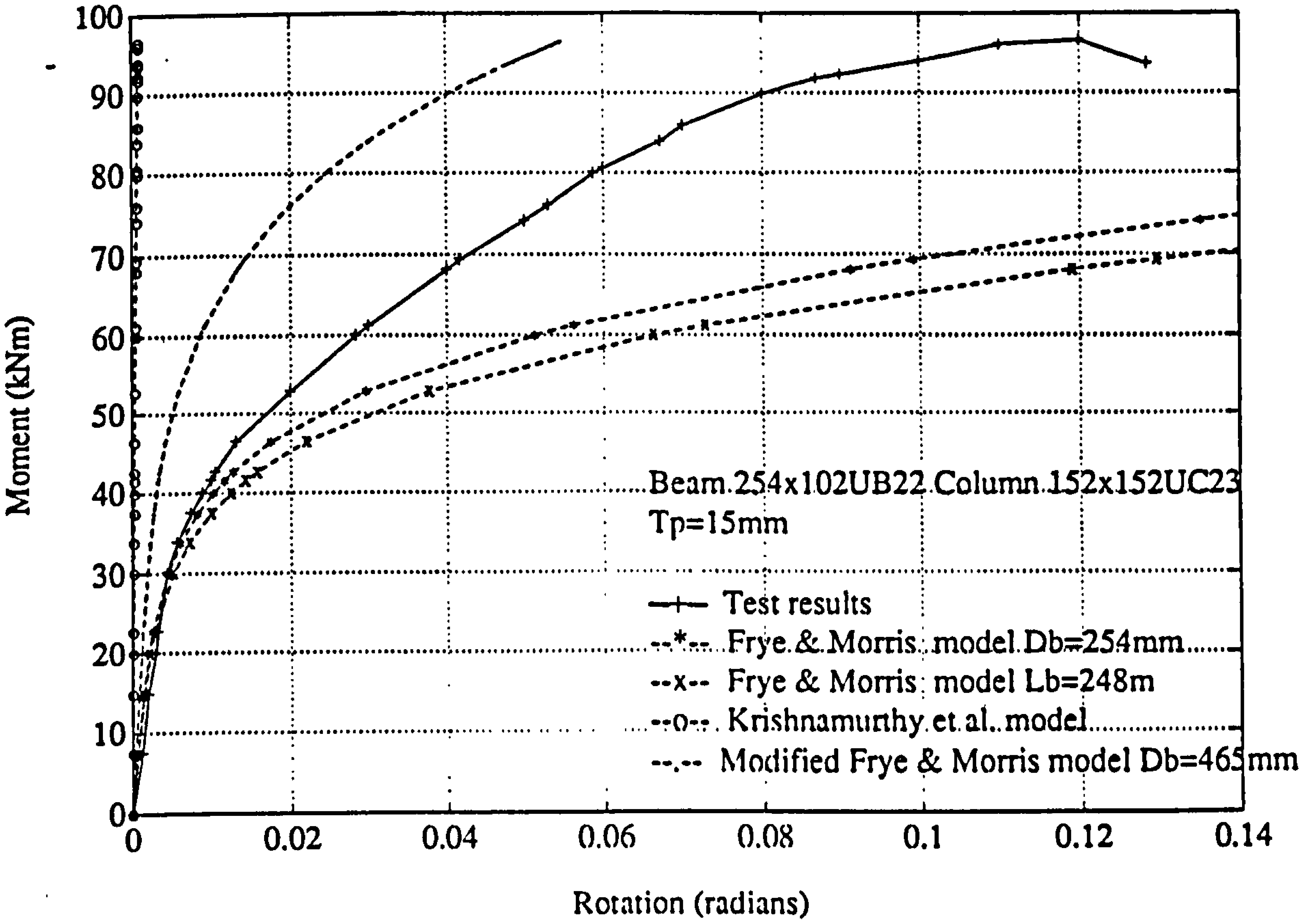


FIG.4-19 Comparison between analytical moment-rotation relationships and Moore & Sims test J4 (stiffened column).

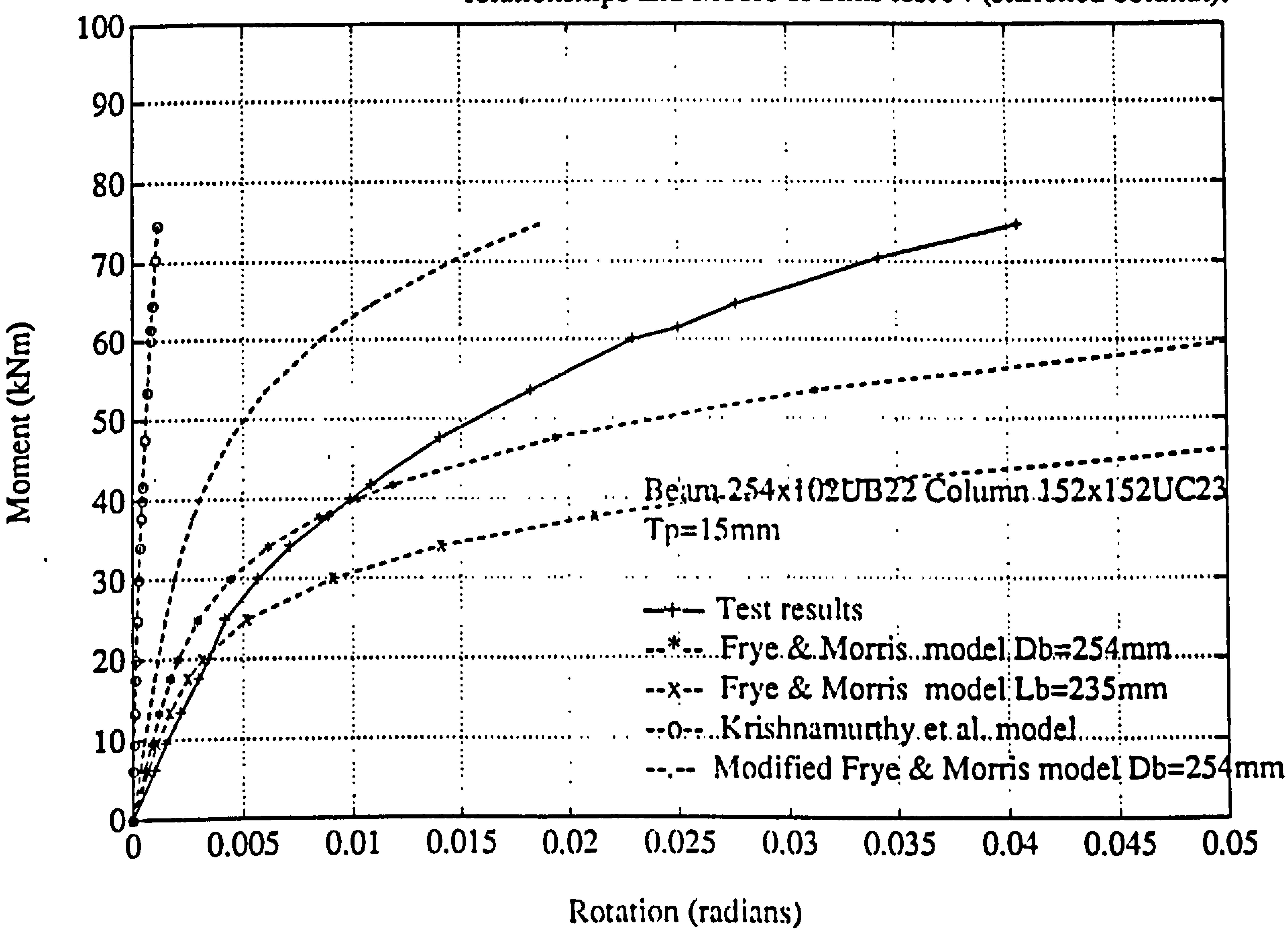


FIG.4-20 Comparison between analytical moment-rotation relationships and Tong test 2 (stiffened column)

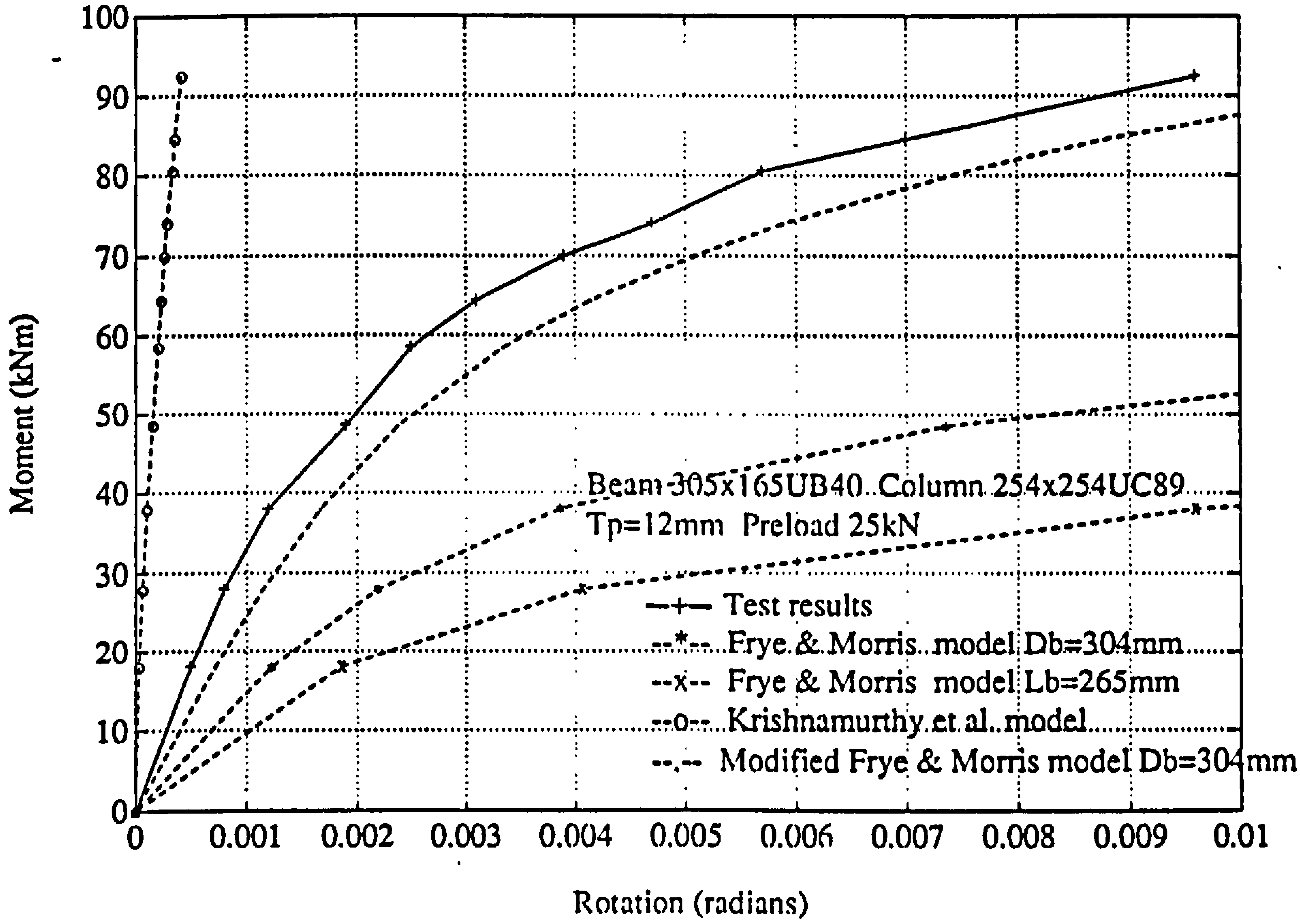


FIG.4-21 Comparison between analytical moment-rotation relationships and Tong test 4 (stiffened column)

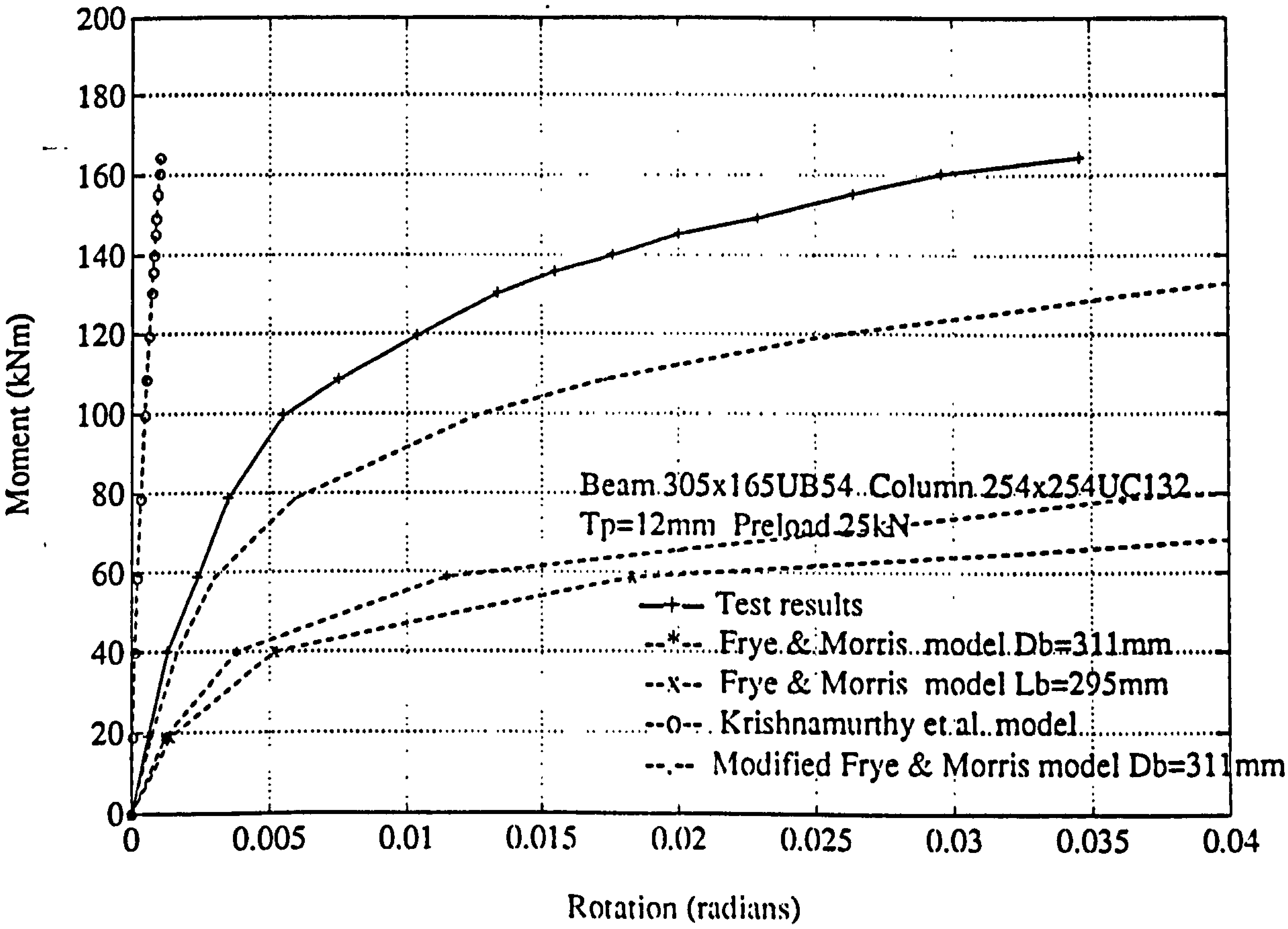


FIG.4-22 Comparison between analytical moment-rotation relationships and Tong test 7 (stiffened column).

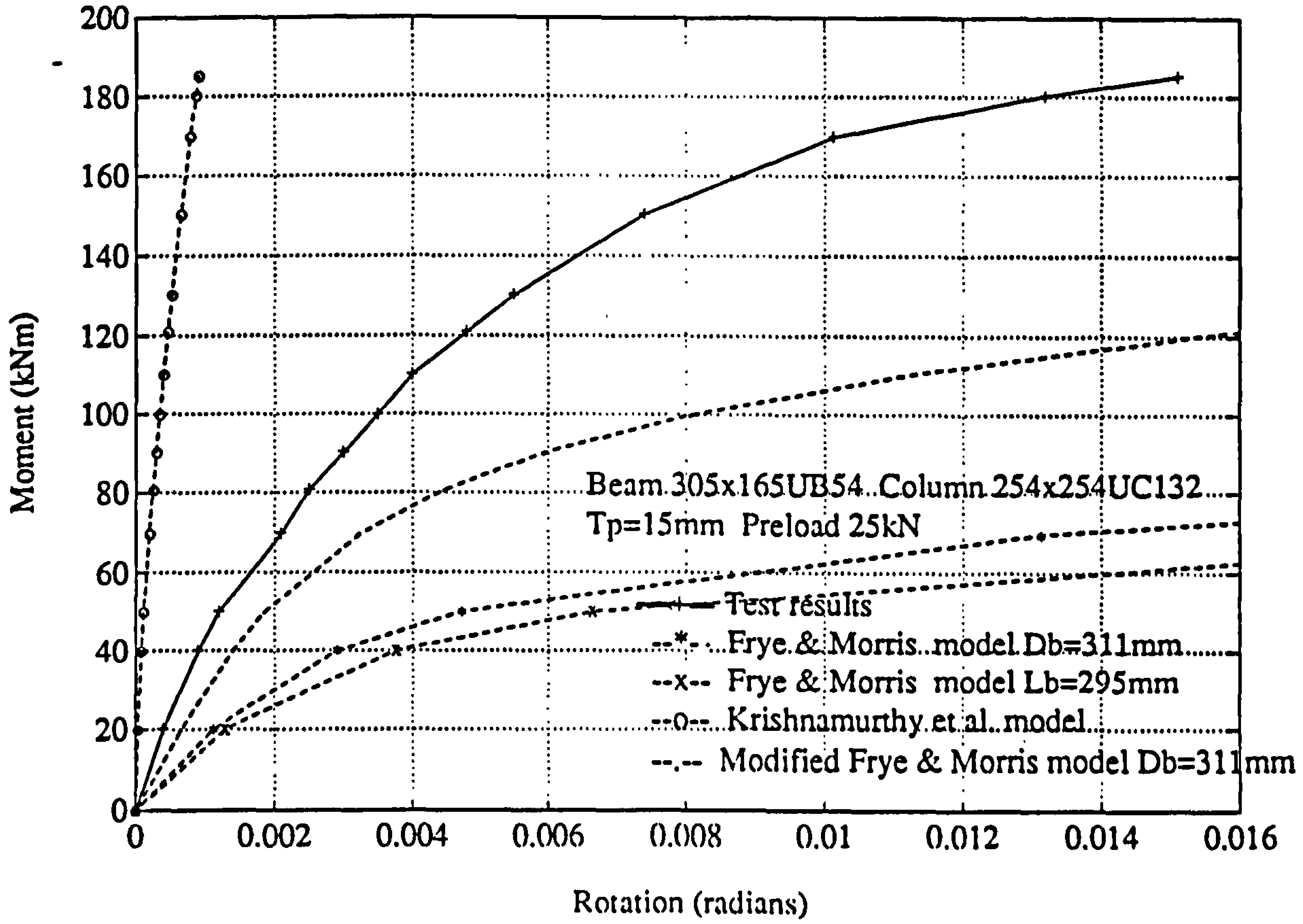


FIG.4-23 Comparison between analytical moment-rotation relationships and Tong test 8 (stiffened column)

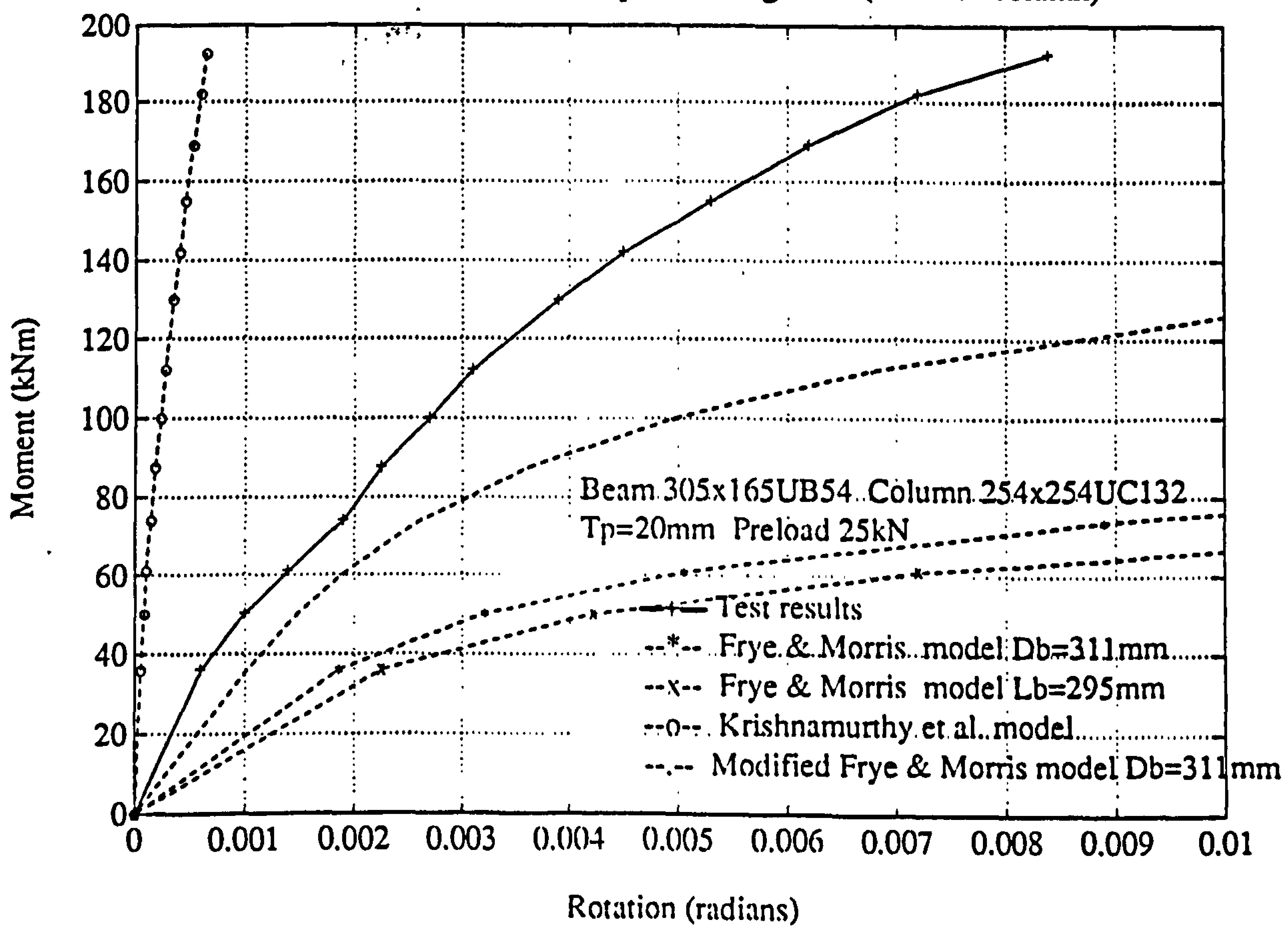


FIG.4-24 Comparison between analytical moment-rotation relationships and Tong test 9 (stiffened column)

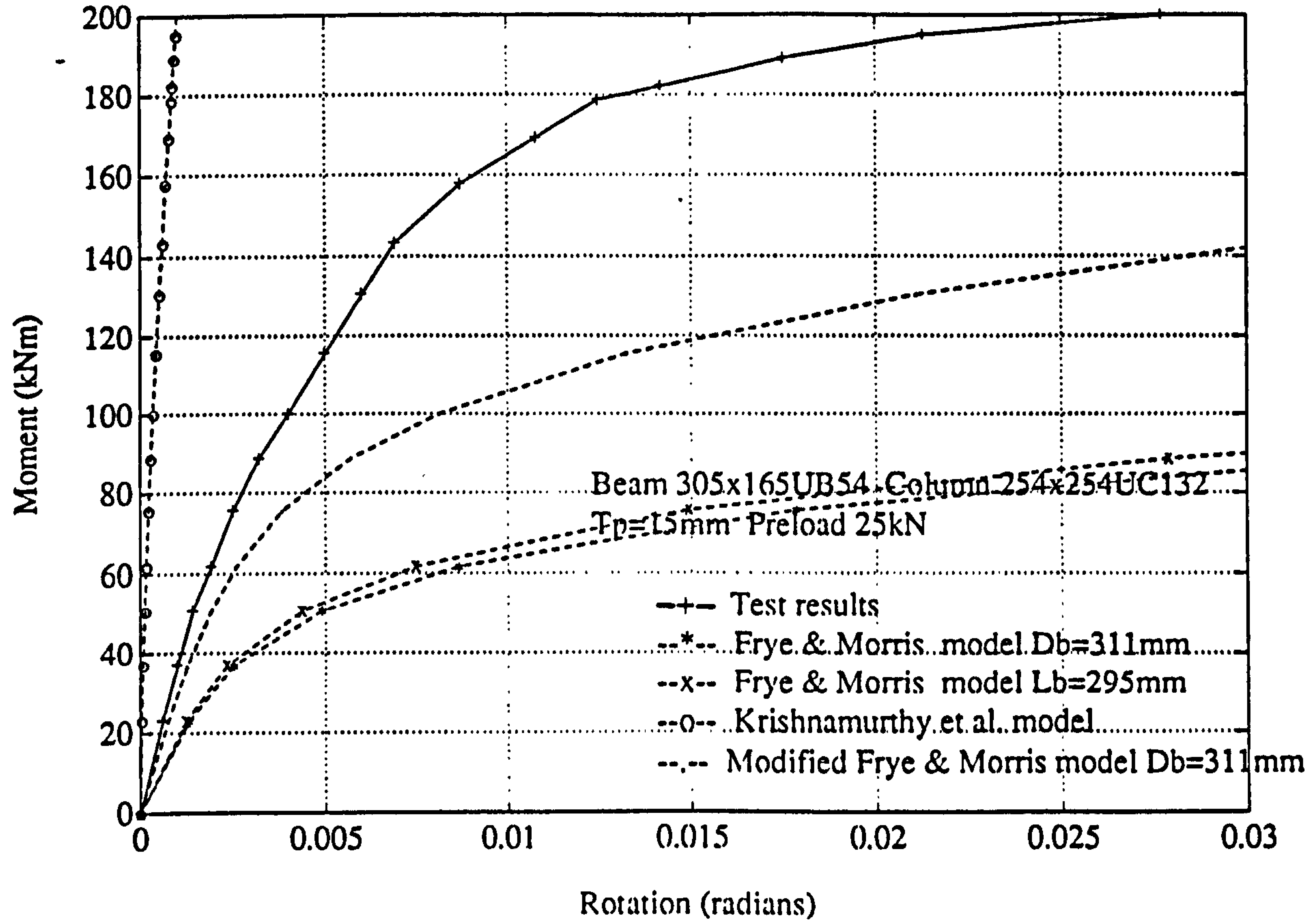


FIG.4-25 Comparison between analytical moment-rotation relationships and Tong test 10 (stiffened column)

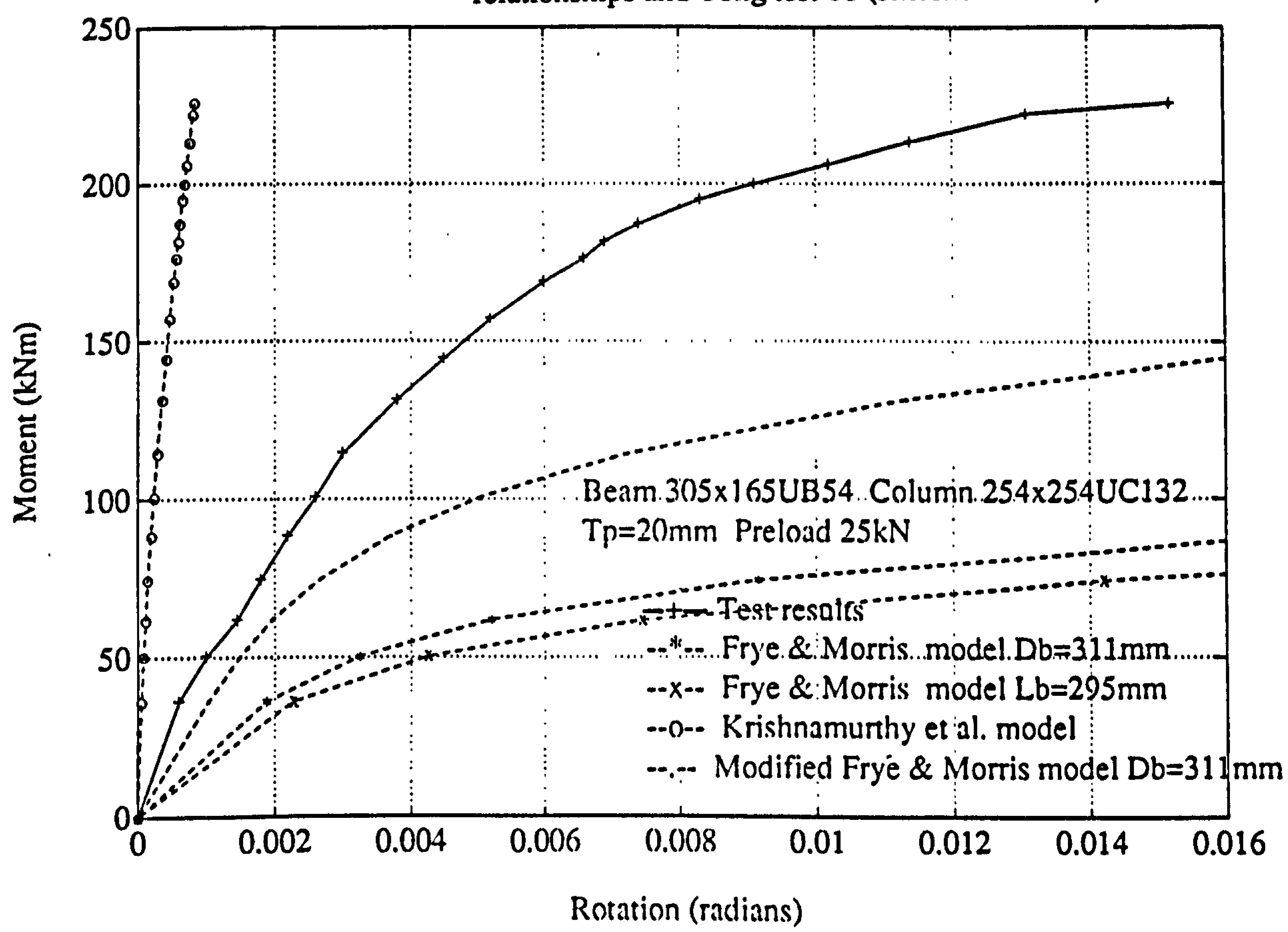


FIG.4-26 Comparison between analytical moment-rotation relationships and Tong test 11 (stiffened column)

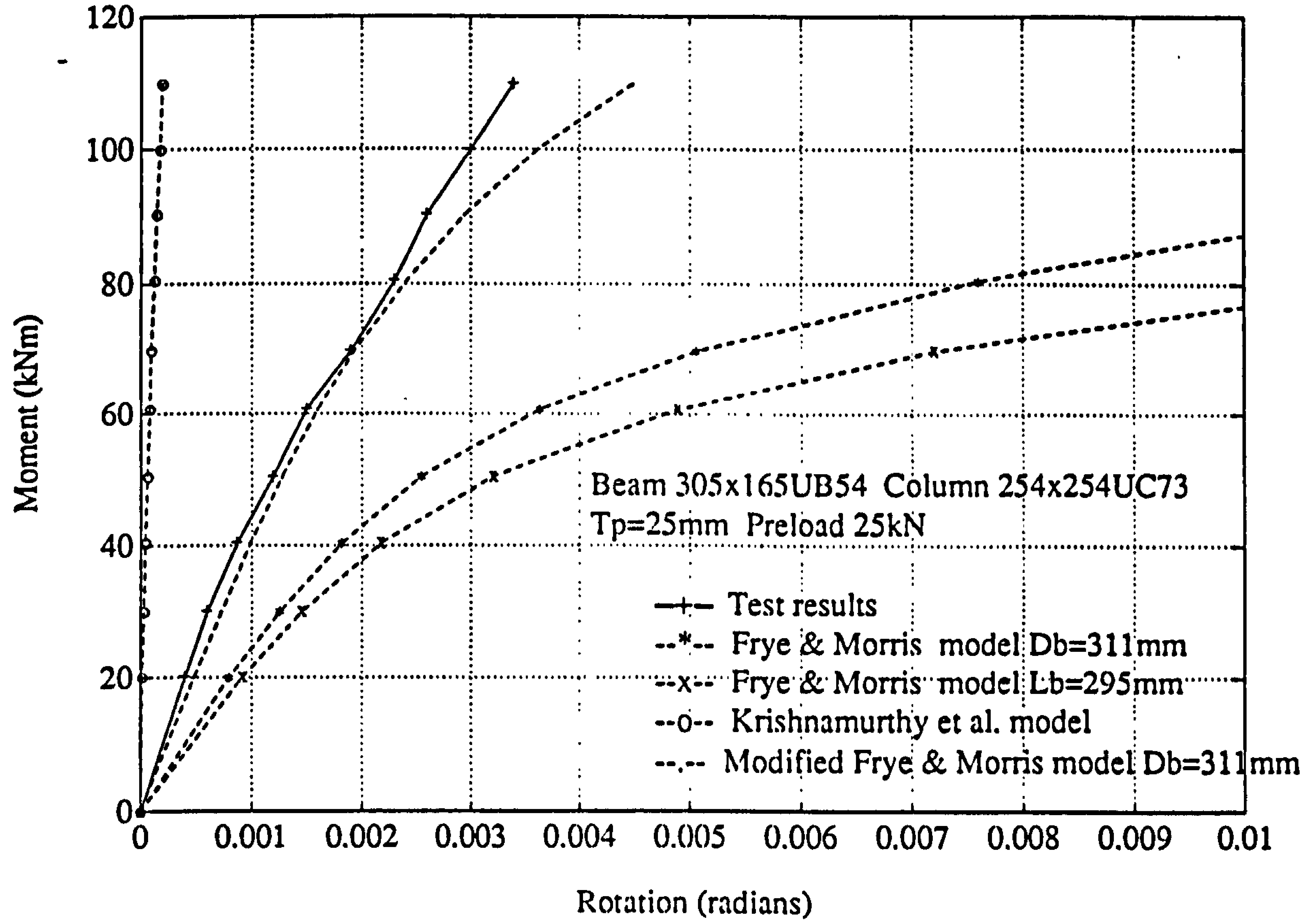


FIG.4-27 Comparison between analytical moment-rotation relationships and Tong test 13 (stiffened column).

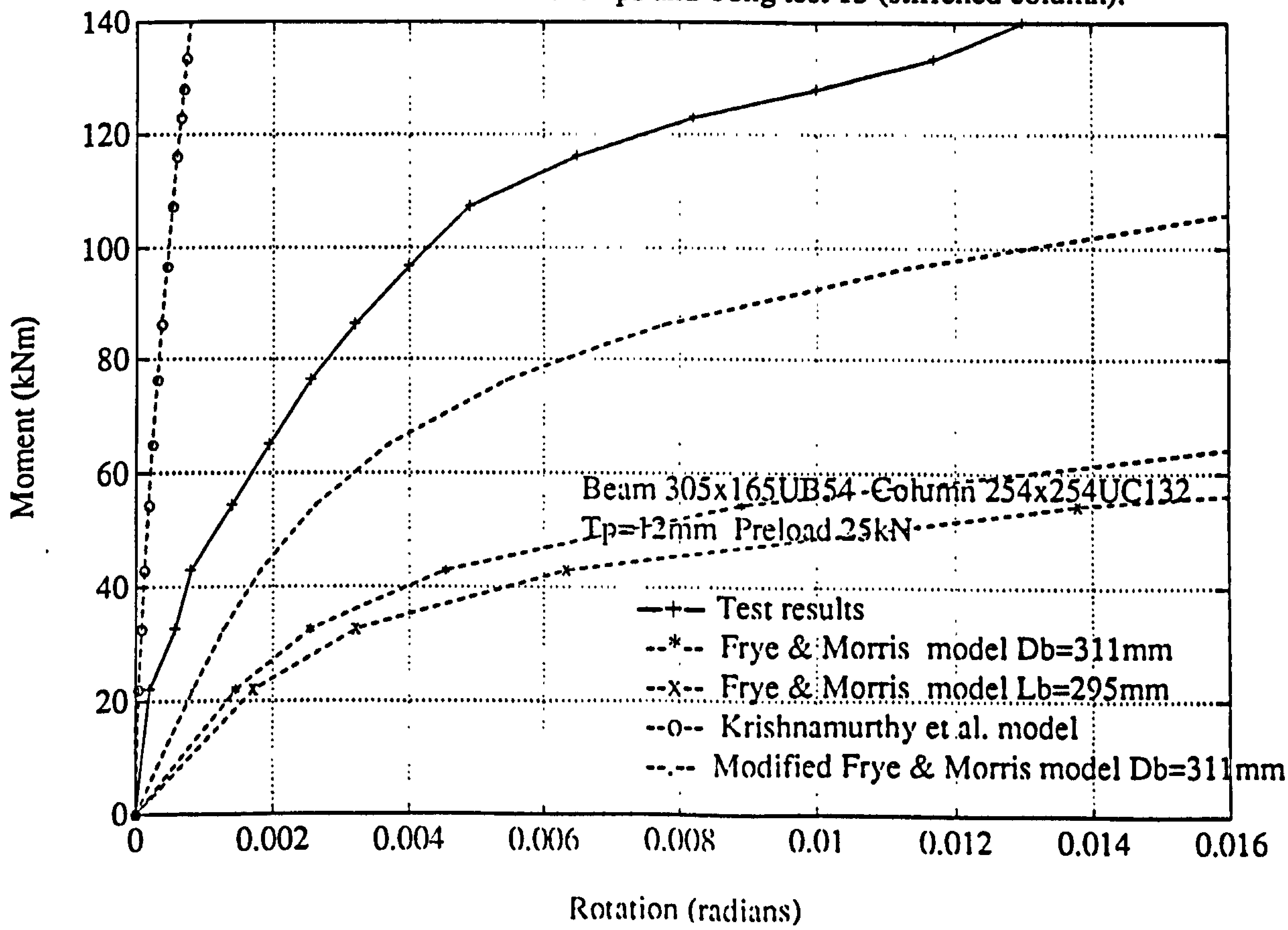


FIG.4-28 Comparison between analytical moment-rotation relationships and Tong test 14 (stiffened column).

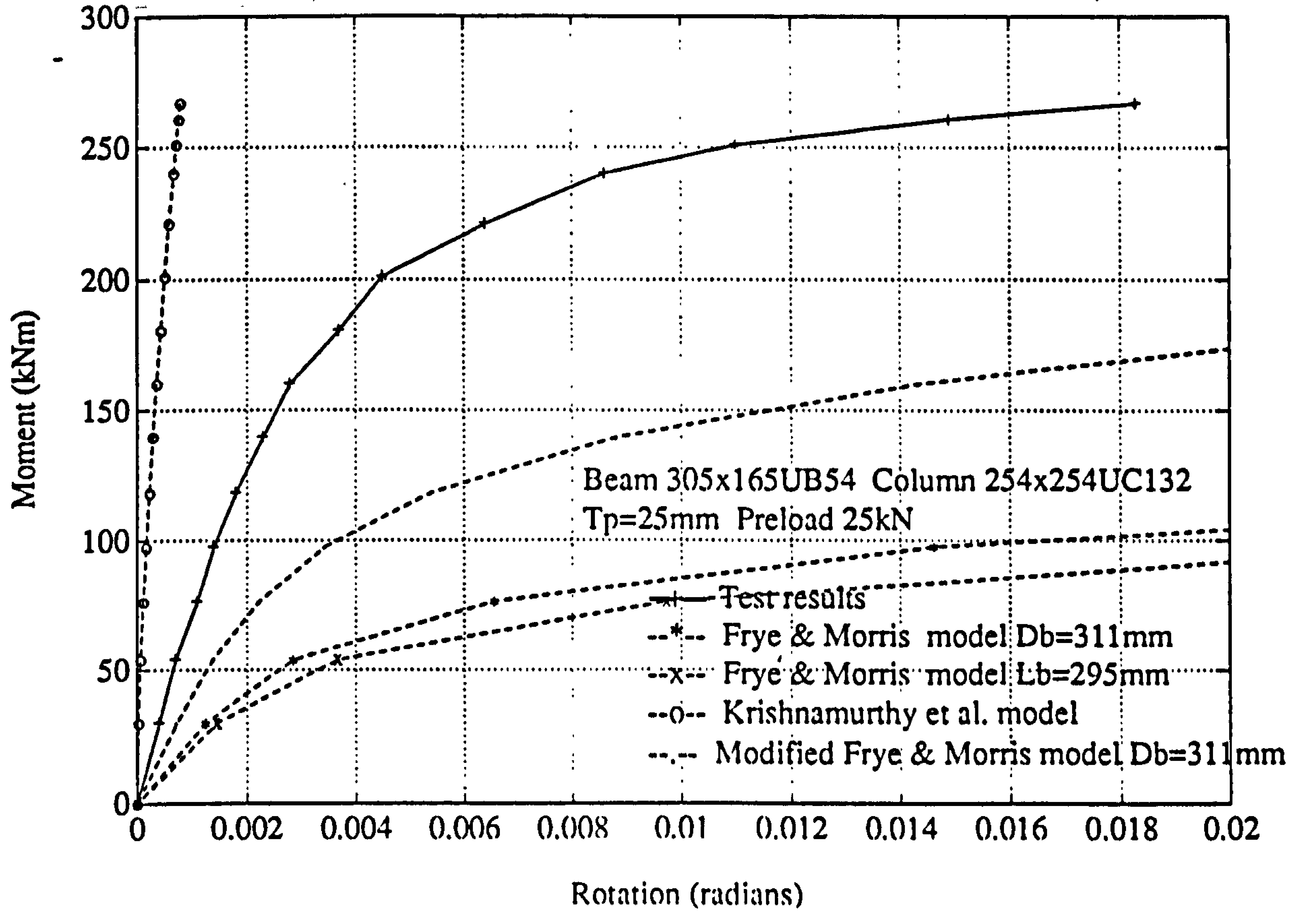


FIG.4-29 Comparison between analytical moment-rotation relationships and Tong test 15 (stiffened column)

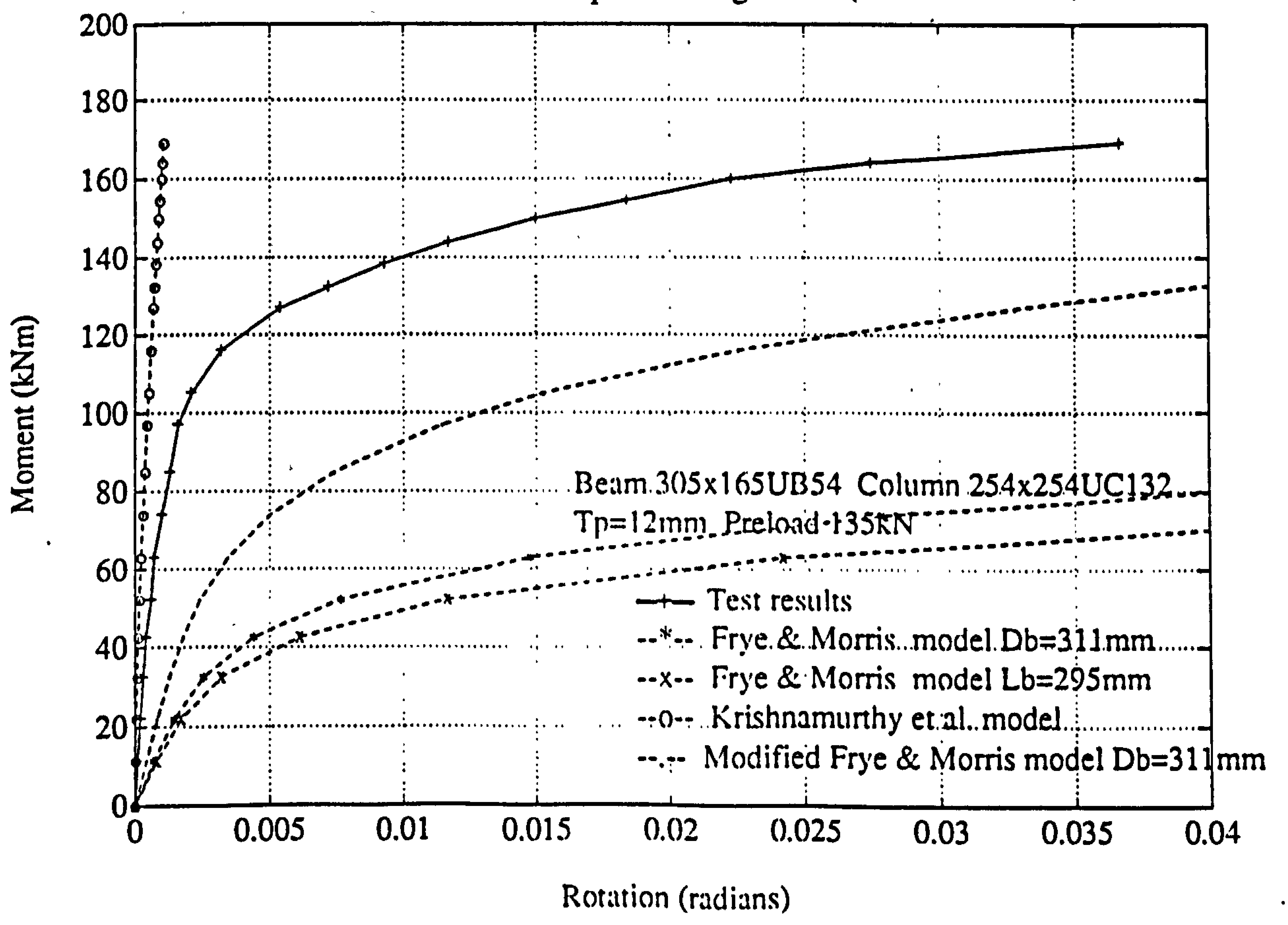


FIG.4-30 Comparison between analytical moment-rotation relationships and Tong test 18 (stiffened column)

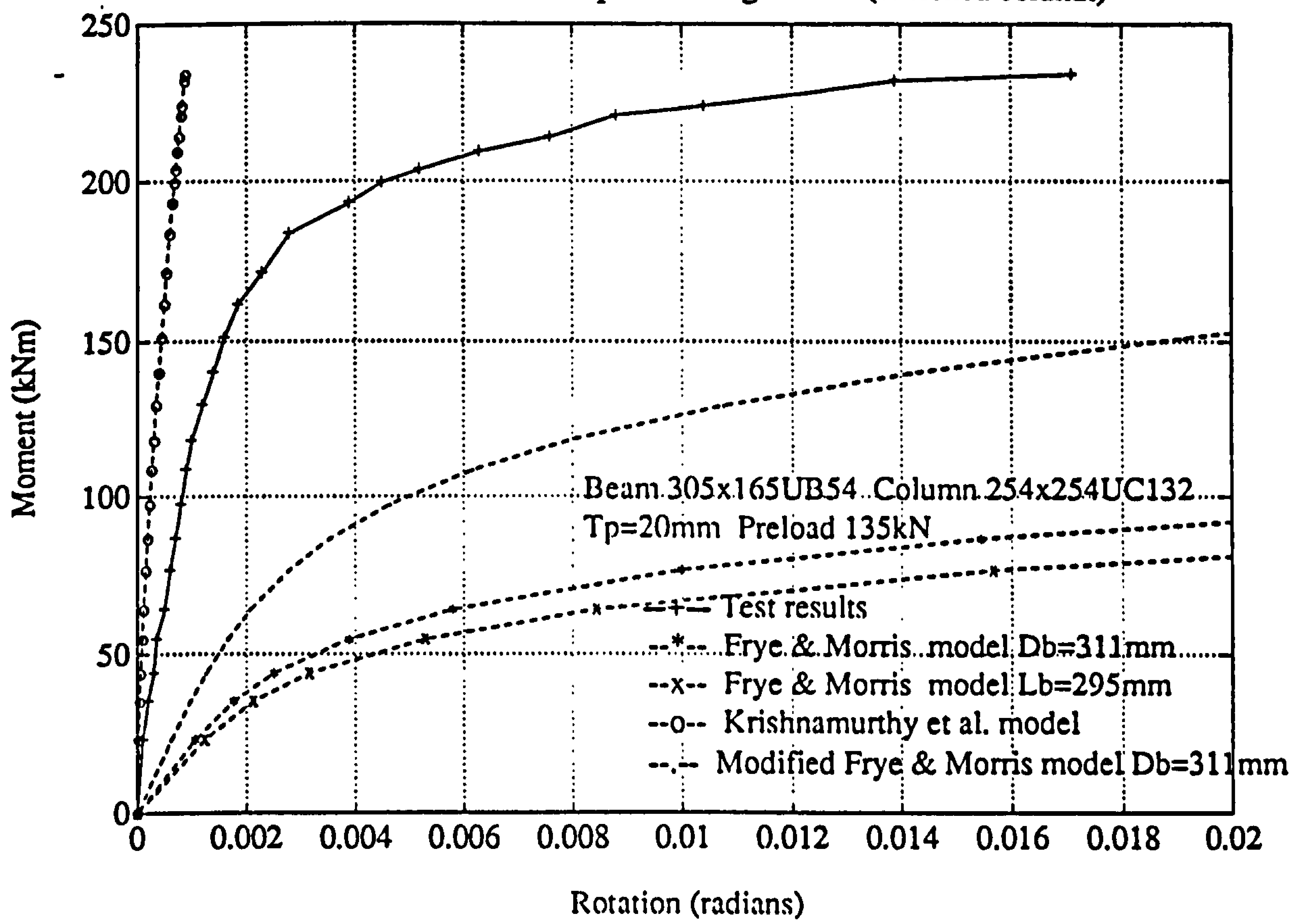


FIG.4-31 Comparison between analytical moment-rotation relationships and Prescott test 22 (stiffened column)

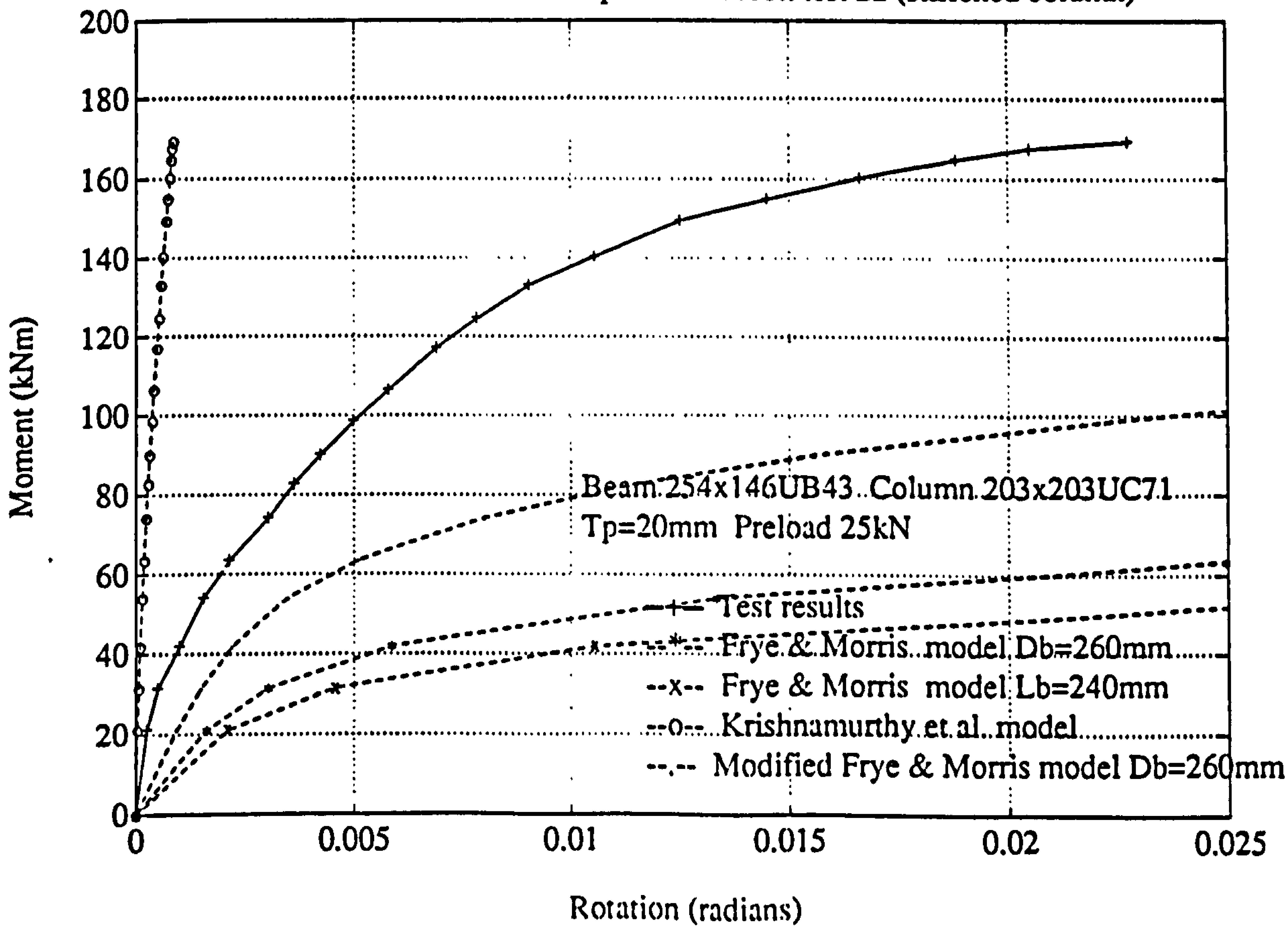


FIG.4-32 Comparison between analytical moment-rotation relationships and Prescott test 23 (stiffened column)

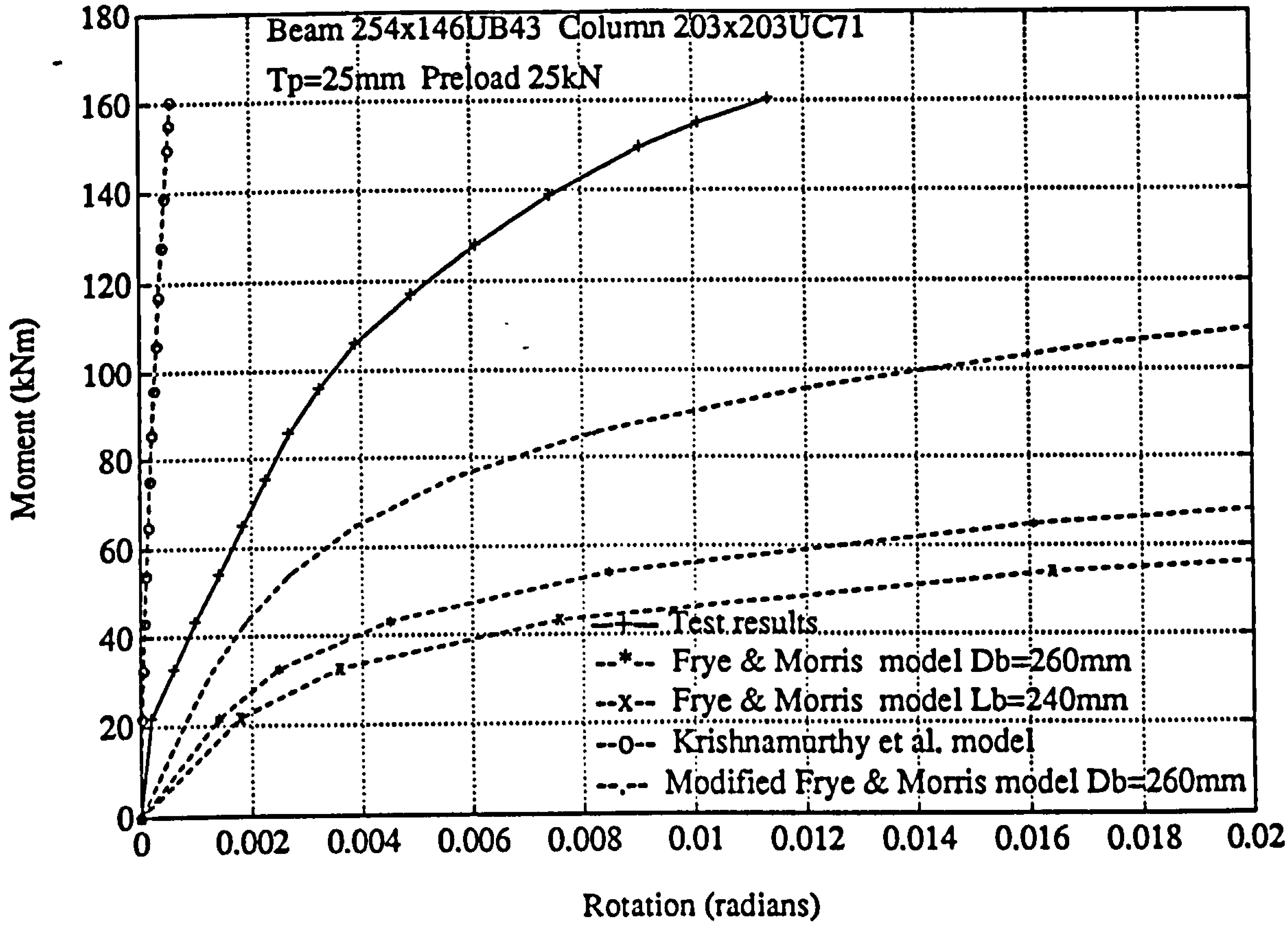


FIG.4-33 Comparison between analytical moment-rotation relationships and Prescott test 27 (stiffened column)

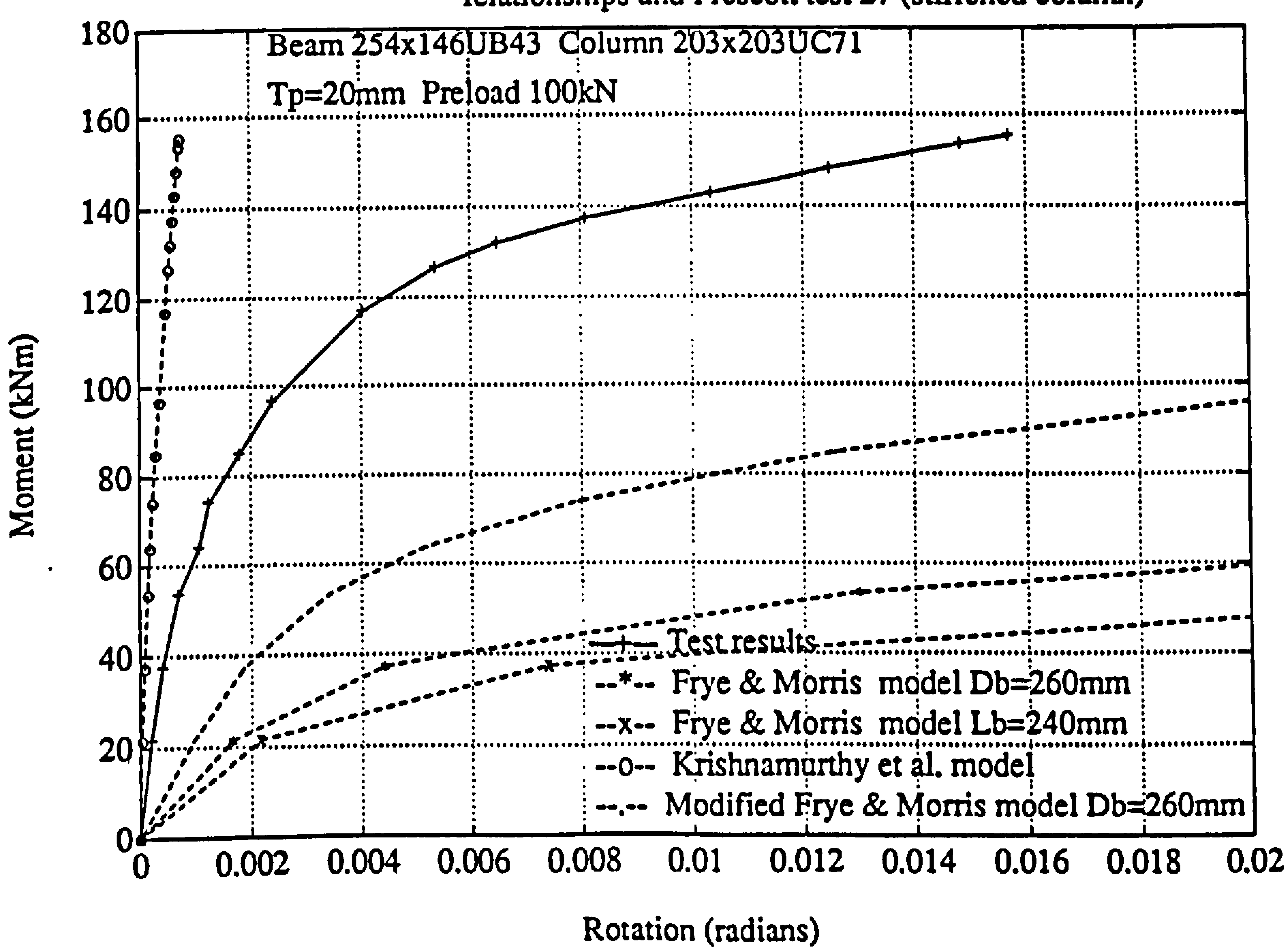


FIG.4-34 Comparison between analytical moment-rotation relationships and Prescott test 28 (stiffened column).

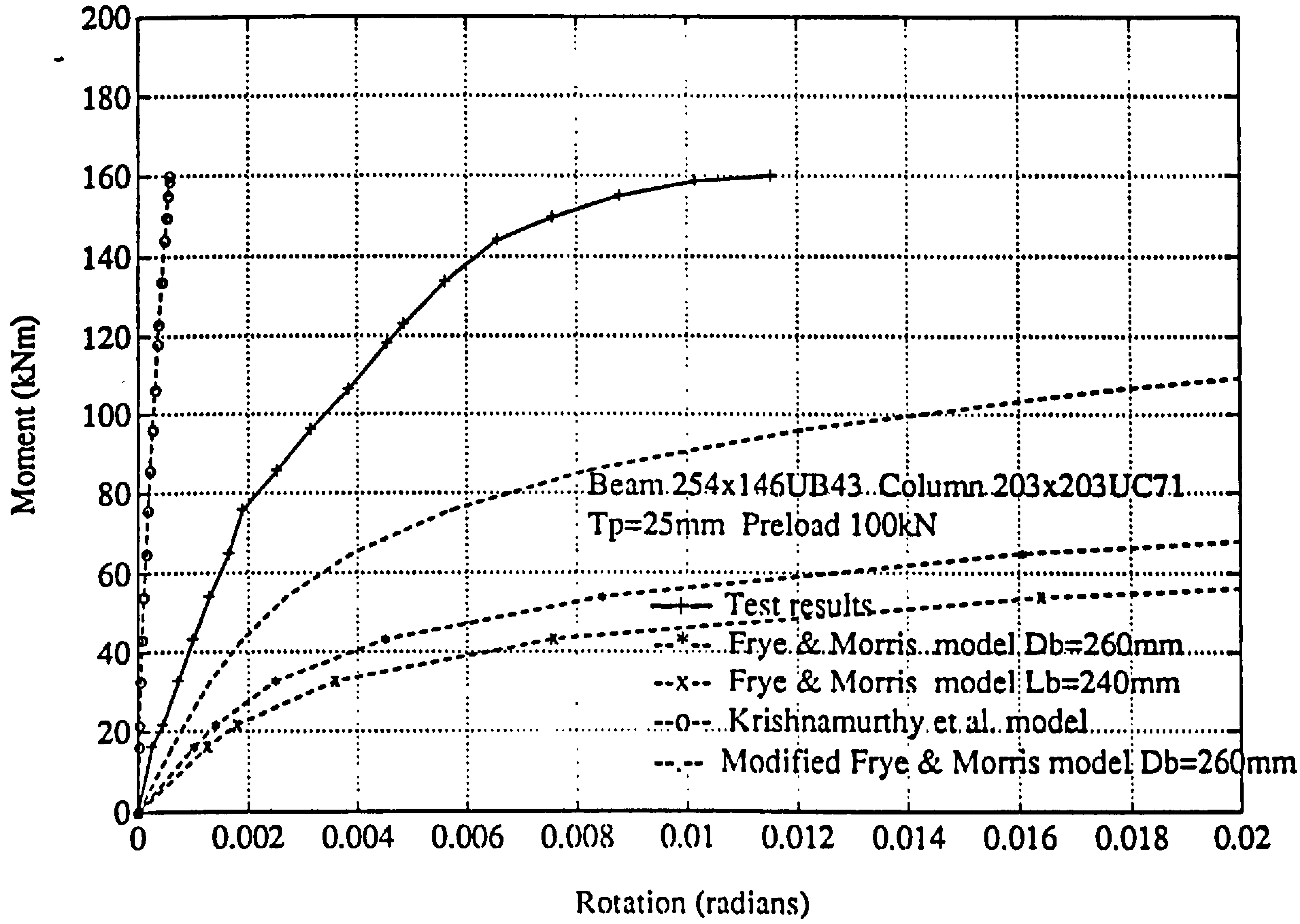


FIG.4-35 Comparison between analytical moment-rotation relationships and Davison test JT/13 (stiffened column)

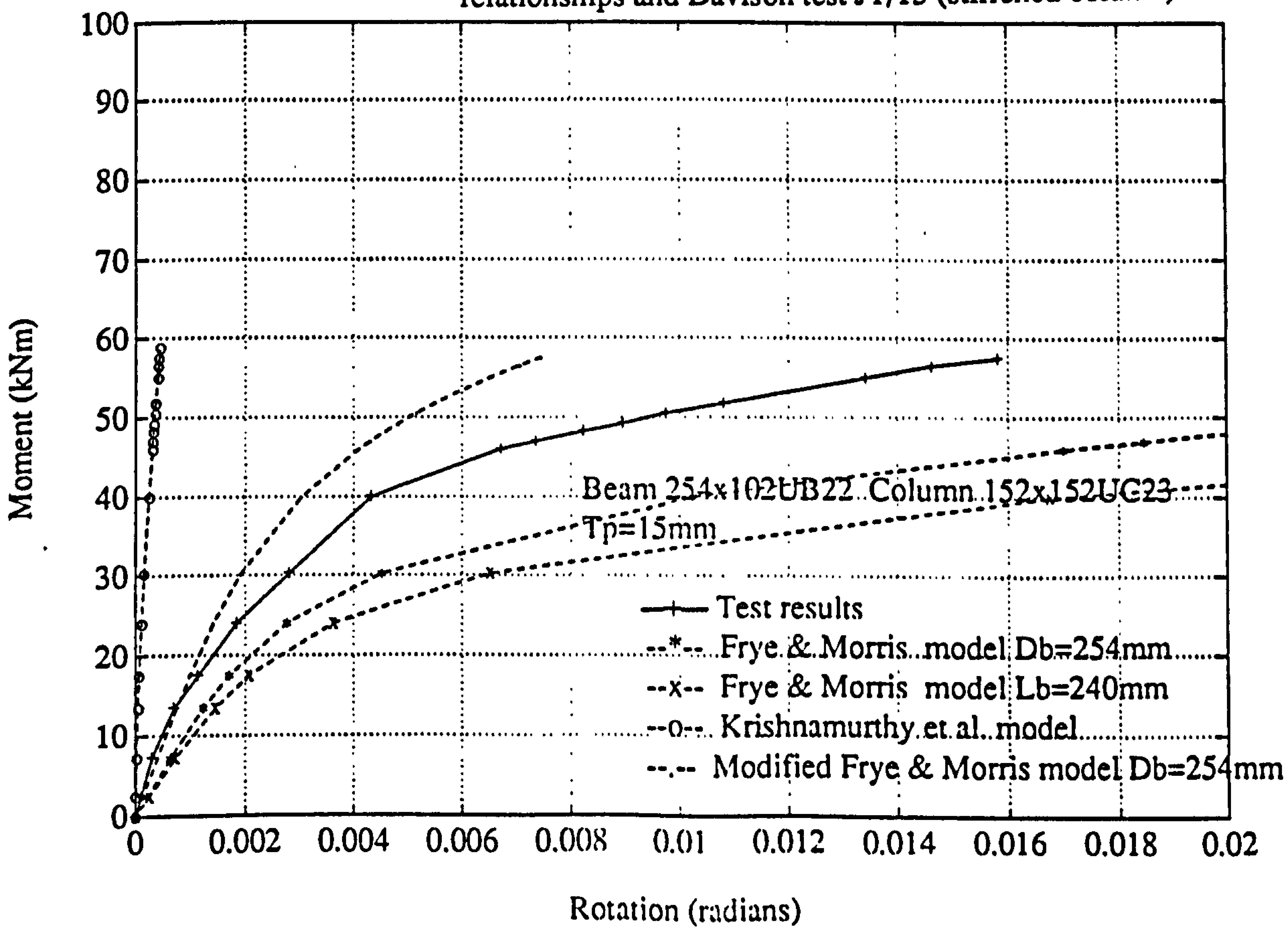


FIG.4-36 Comparison between analytical moment-rotation relationships and Bailey test B4 (stiffened column)

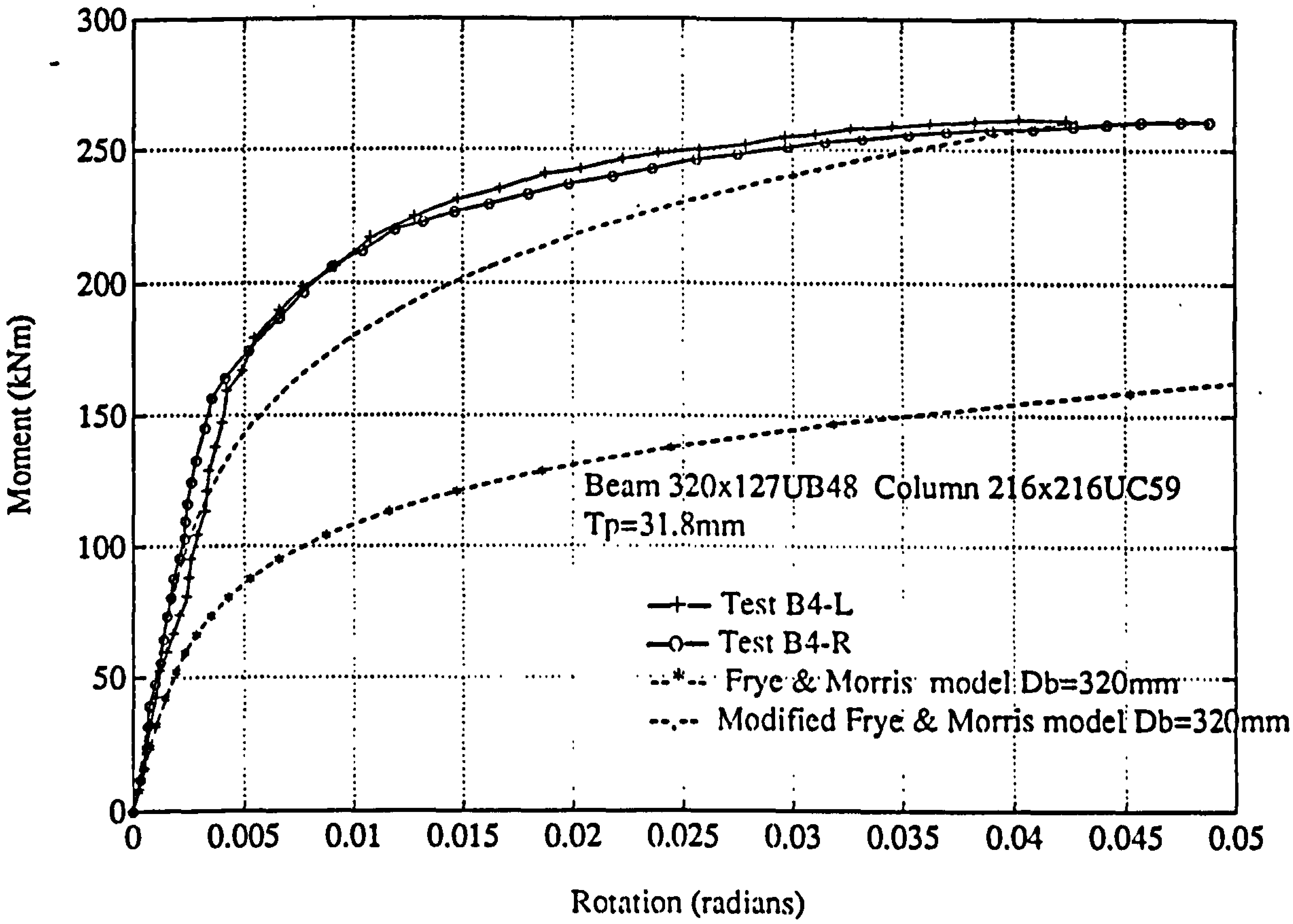


FIG.4-37 Comparison between analytical moment-rotation relationships and Bailey test B5 (stiffened column)

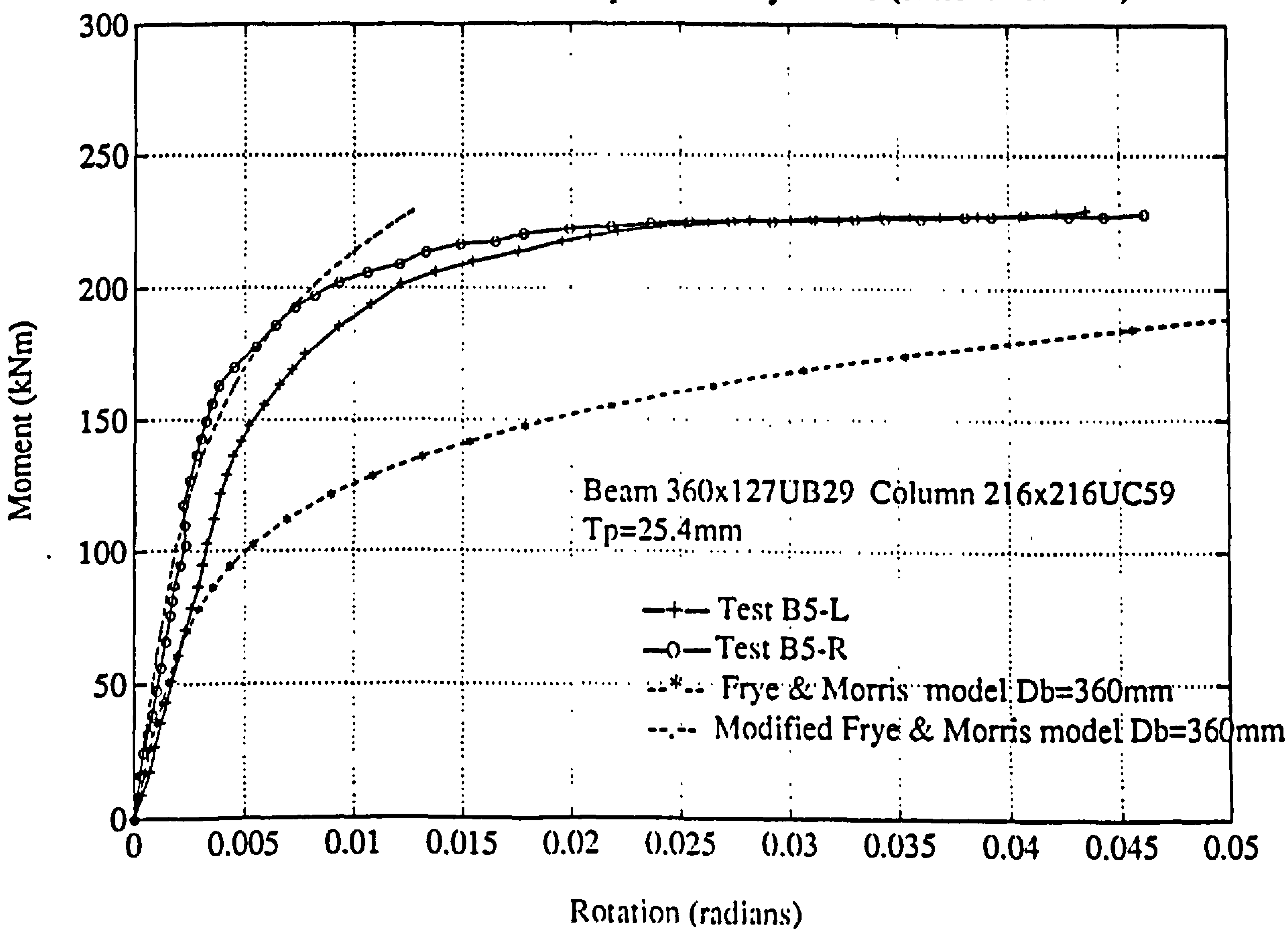


FIG.4-38 Comparison between analytical moment-rotation relationships and Bailey test B6 (stiffened column)

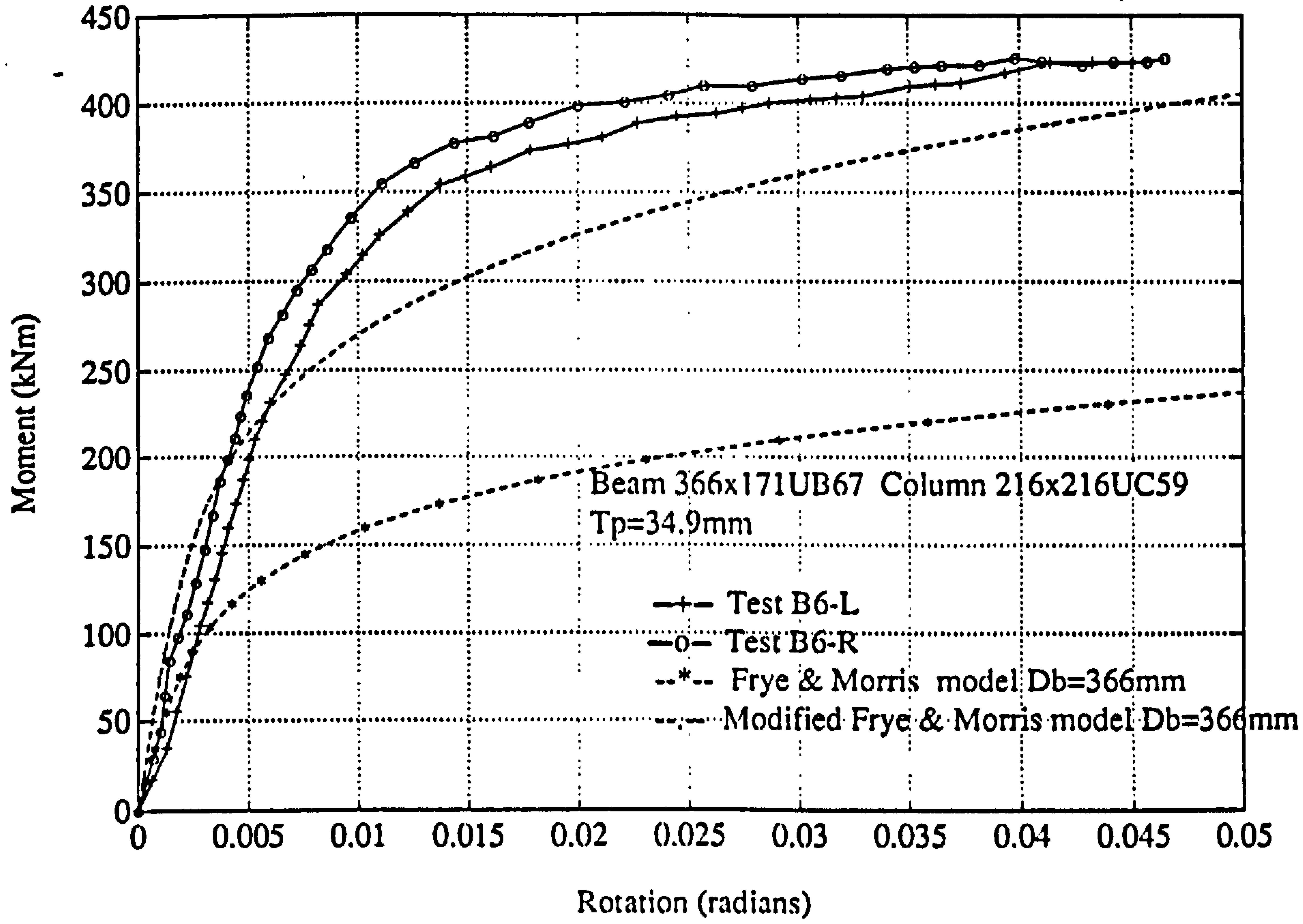


FIG.4-39 Comparison between analytical moment-rotation relationships and Bailey test B7 (stiffened column).

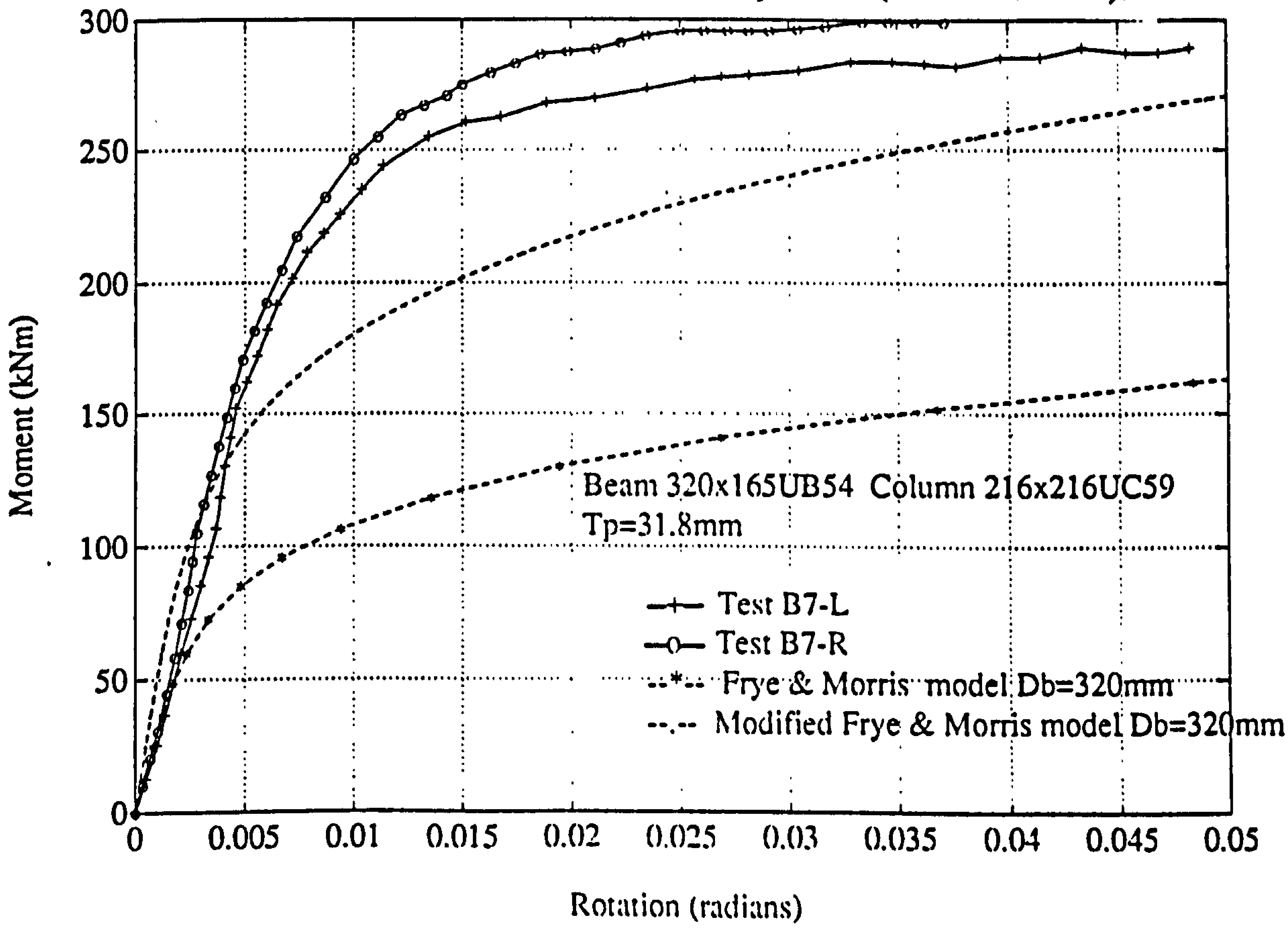


FIG.4-40 Comparison between analytical moment-rotation relationships and Bailey test B8 (stiffened column).

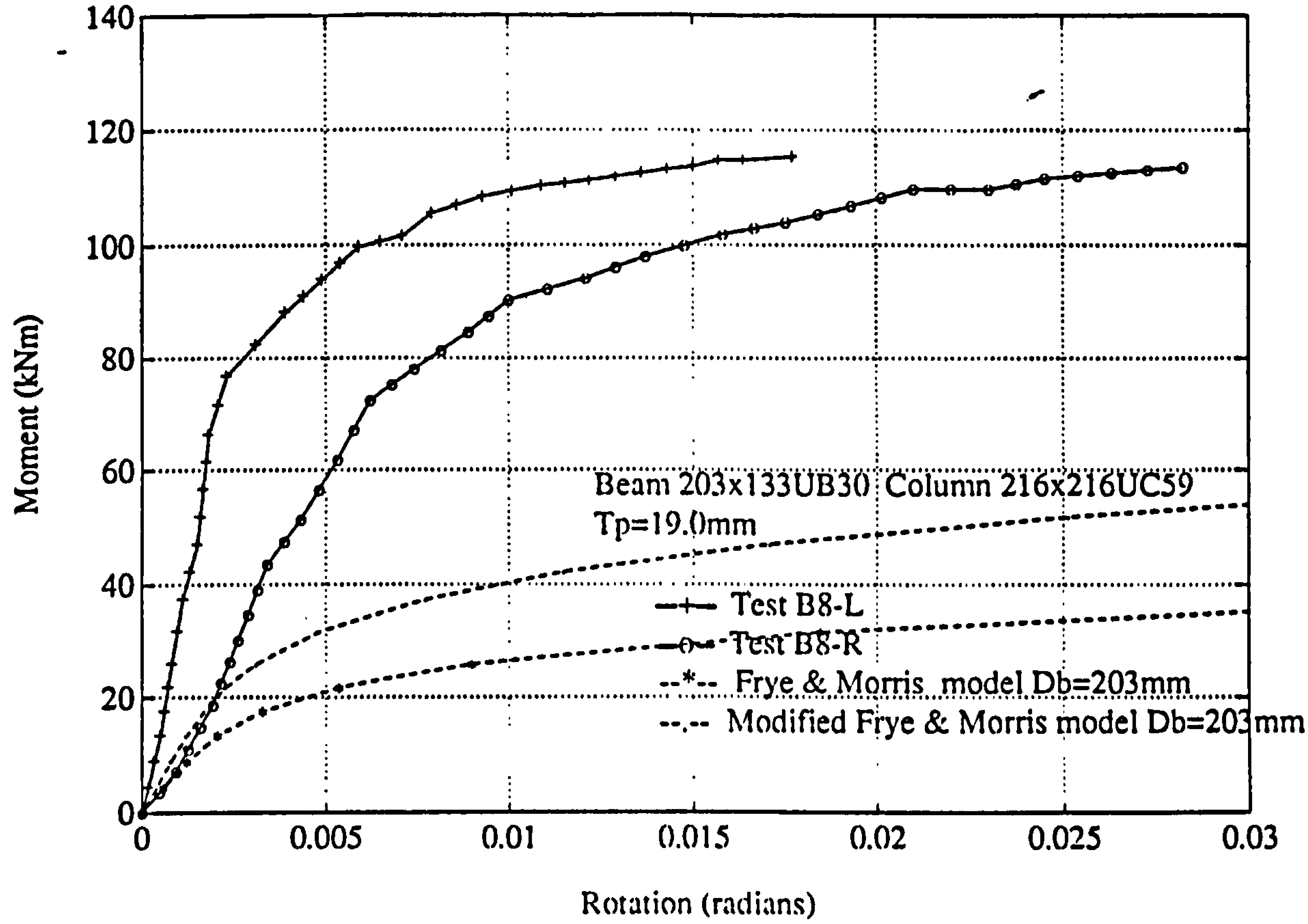


FIG.4-41 Comparison between analytical moment-rotation relationships and Bailey test C9 (stiffened column)

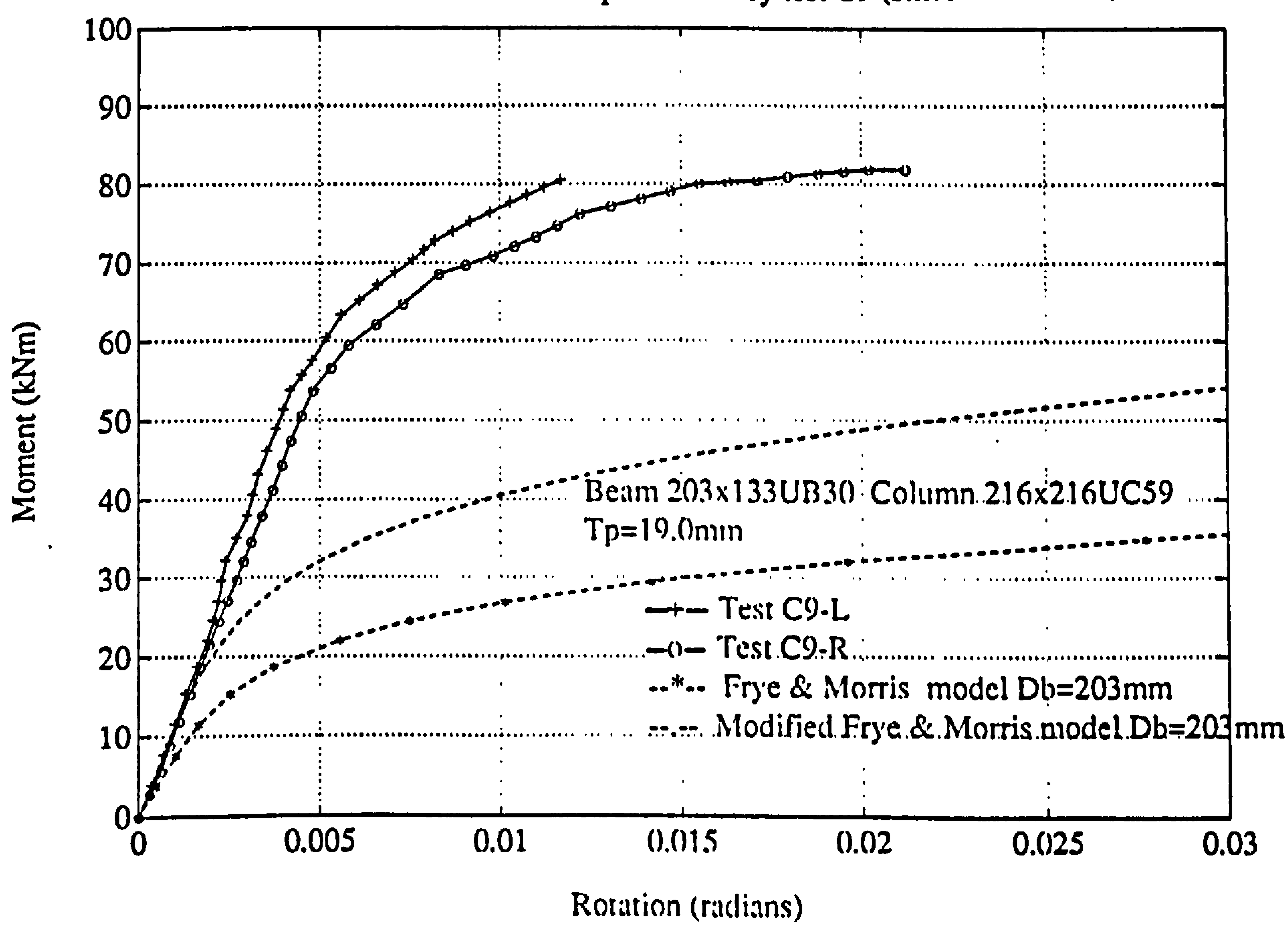


FIG.4-42 Comparison between analytical moment-rotation relationships and Bailey test C10 (stiffened column)

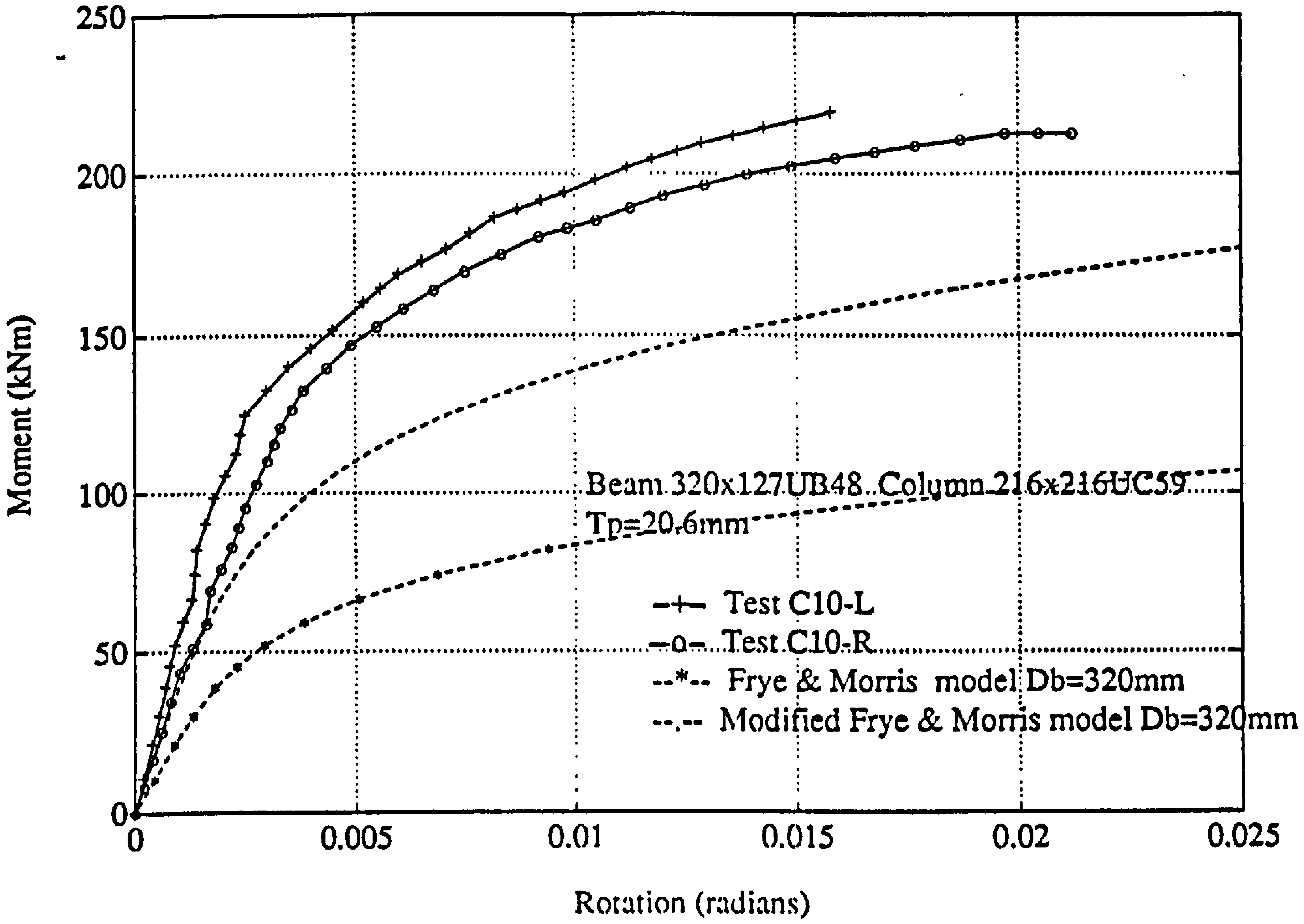


FIG.4-43 Comparison between analytical moment-rotation relationships and Bailey test C11 (stiffened column)

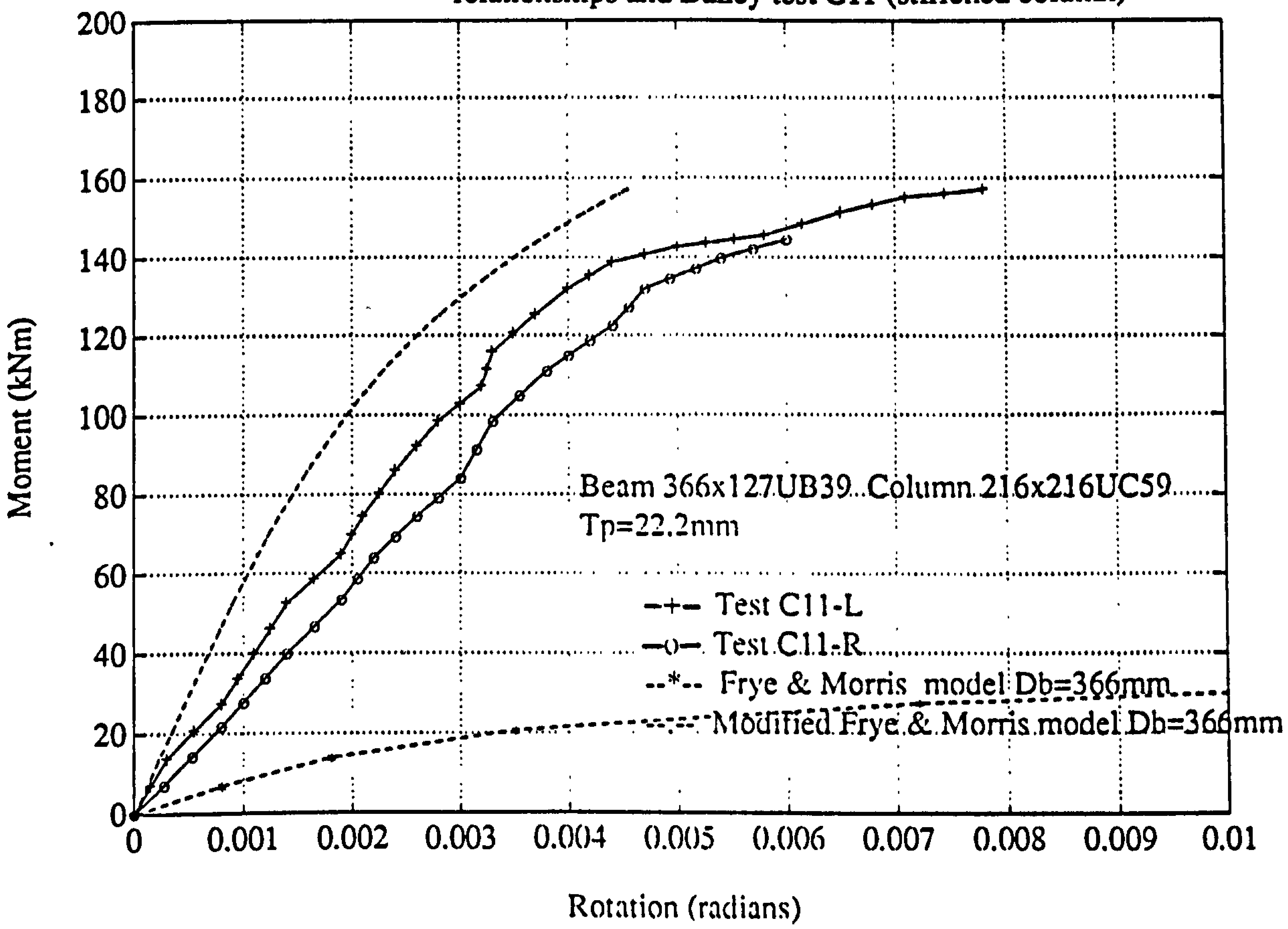


FIG.4-44 Comparison between analytical moment-rotation relationships and Bailey test C12 (stiffened column)

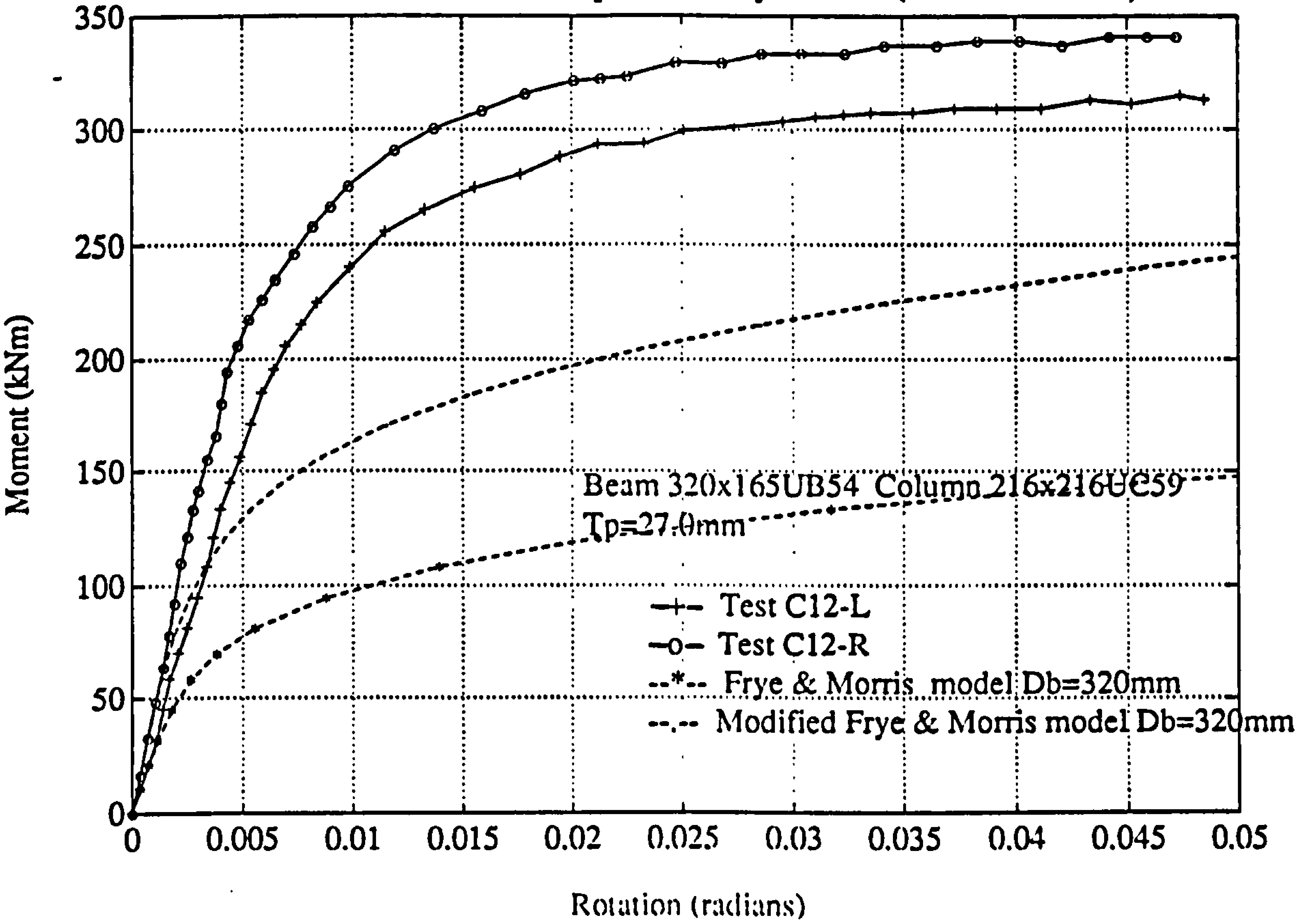


FIG.4-45 Comparison between analytical moment-rotation relationships and Bailey test C13 (stiffened column).

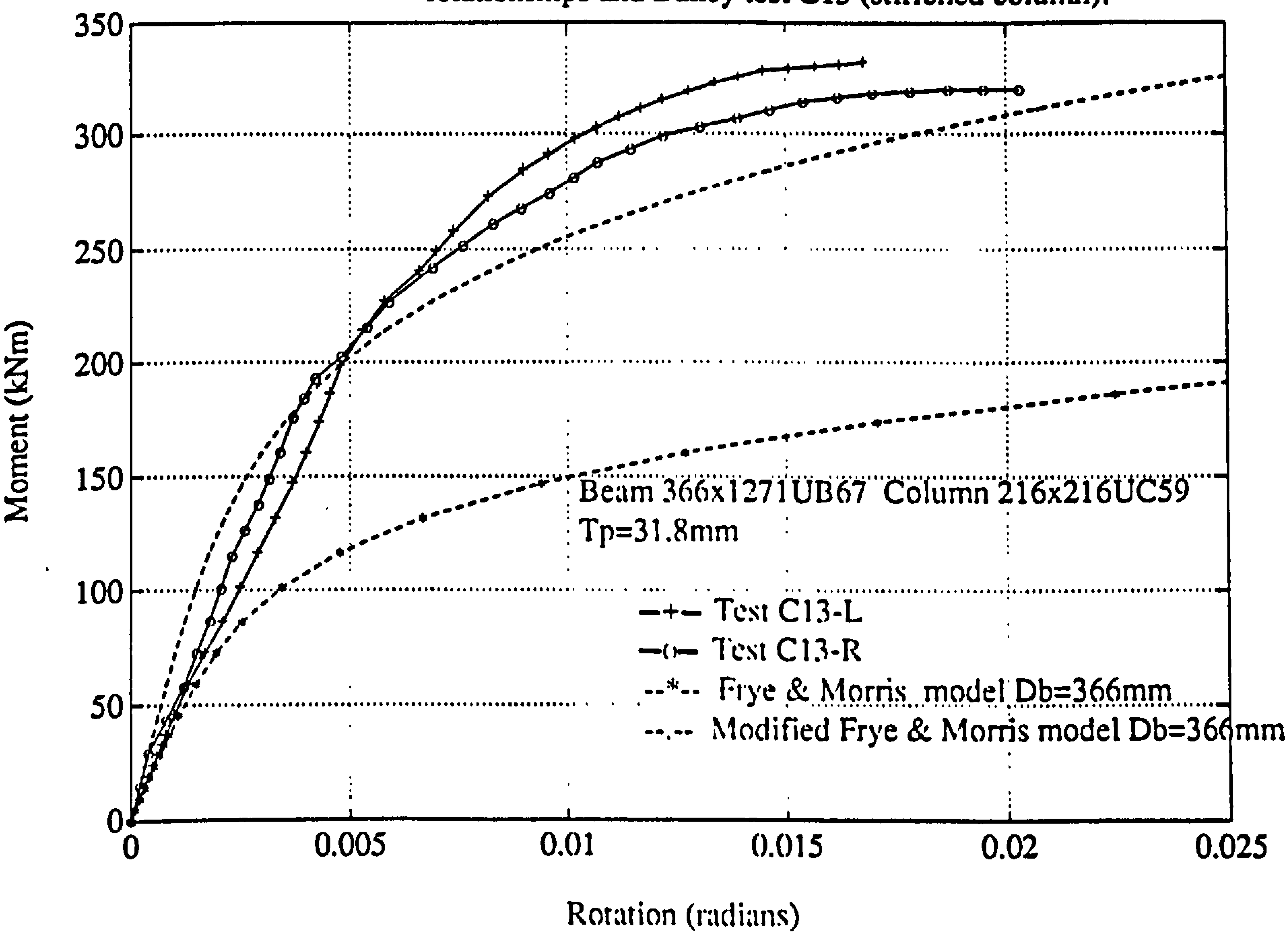


FIG.4-46 Comparison between analytical moment-rotation relationships and Sherbourne test A1 (unstiffened column).

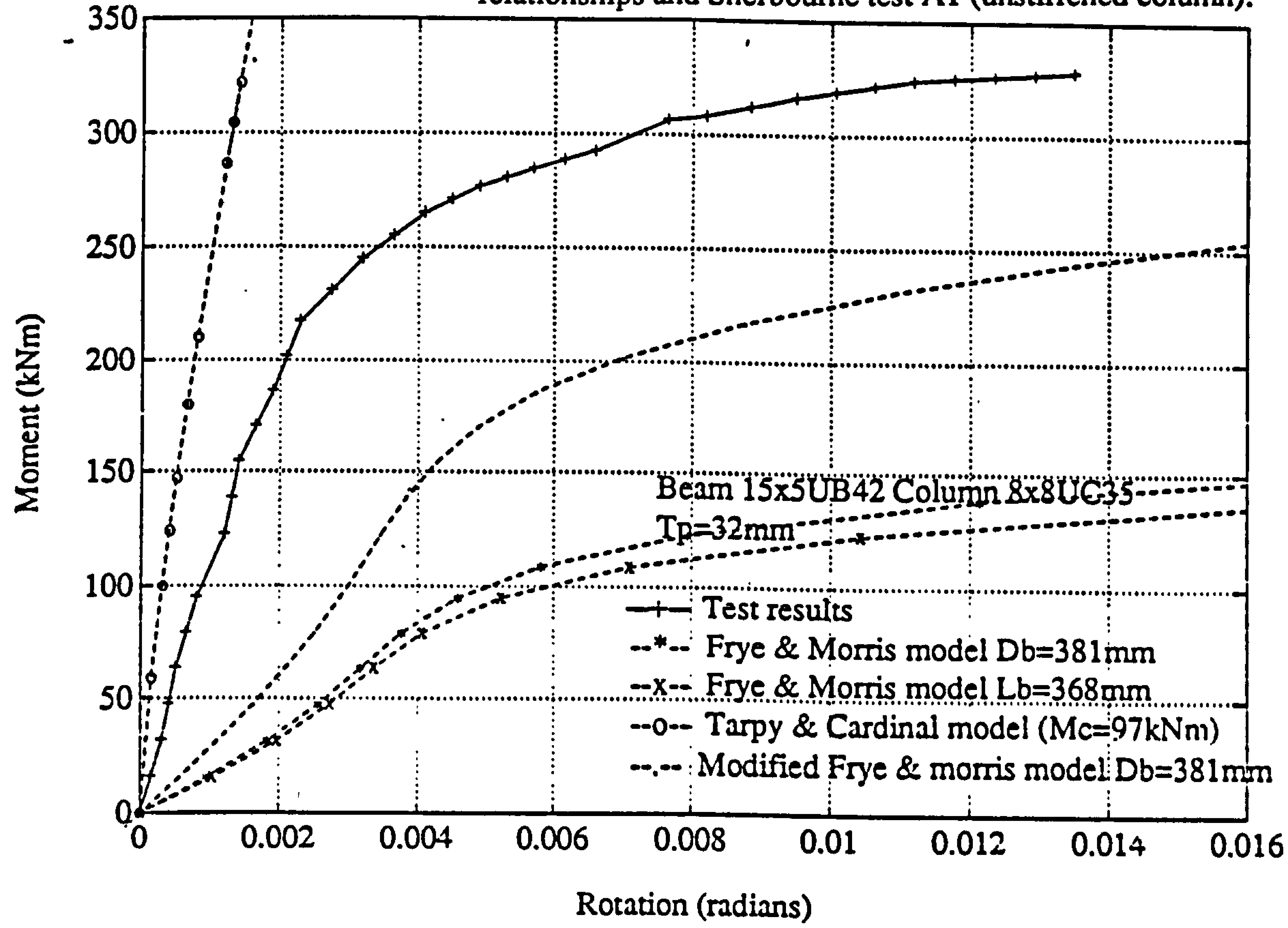


FIG.4-47 Comparison between analytical moment-rotation relationships and Mann test C1 (unstiffened column)

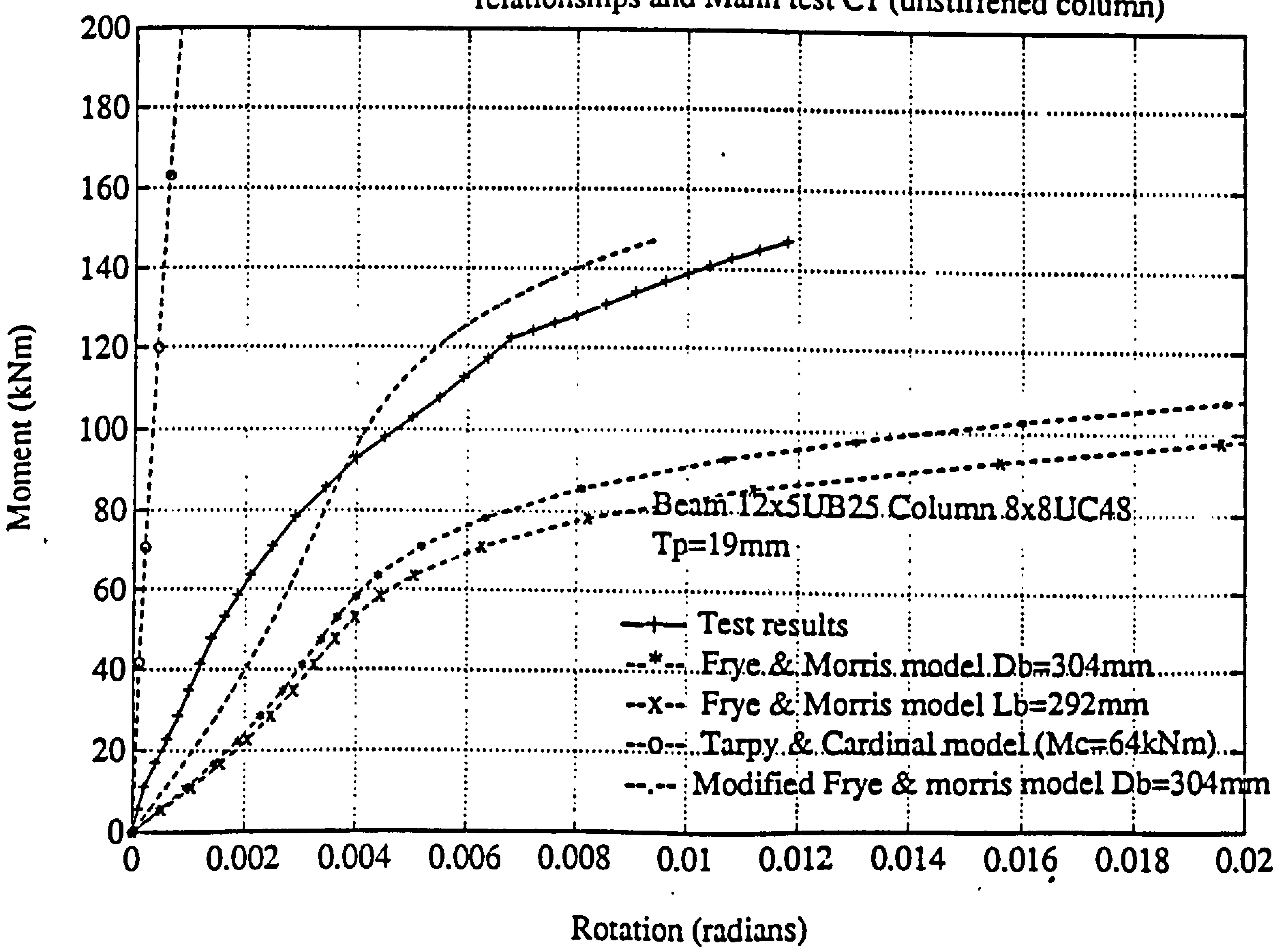


FIG.4-48 Comparison between analytical moment-rotation relationships and Mann test C2 (unstiffened column).

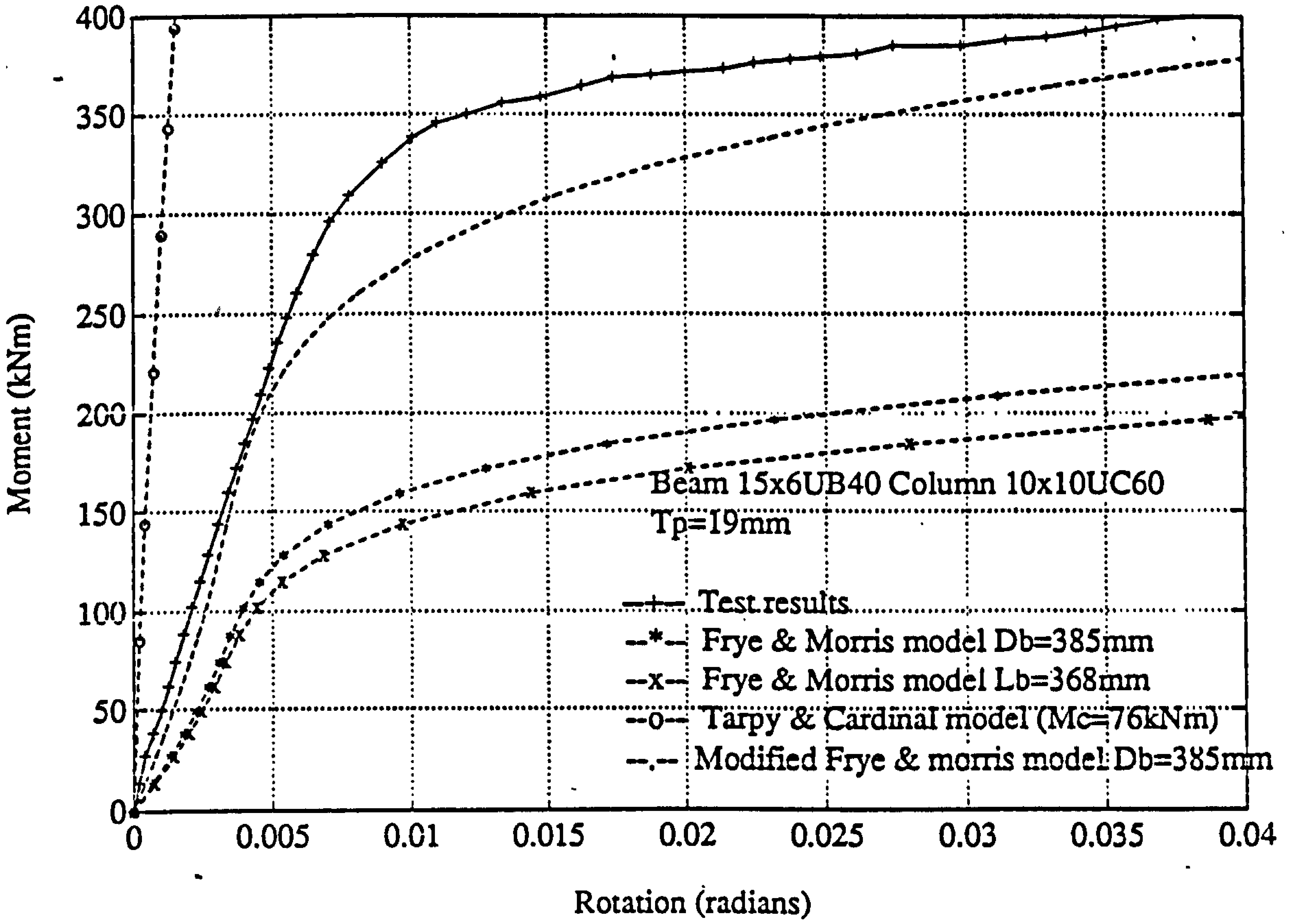


FIG.4-49 Comparison between analytical moment-rotation and Mann test C3 (unstiffened column)

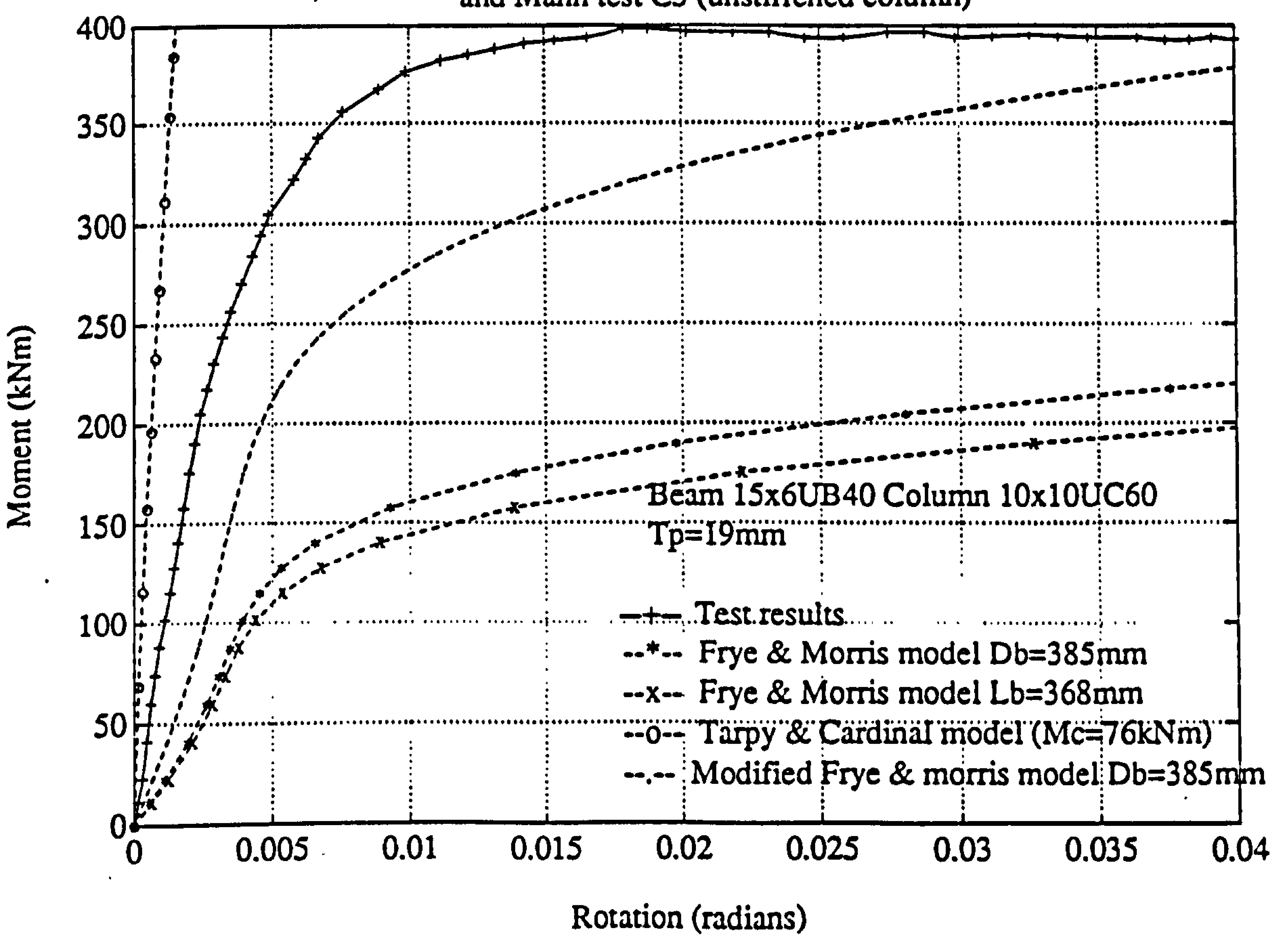


FIG.4-50 Comparison between analytical moment-rotation relationships and Mann test C4 (unstiffened column)

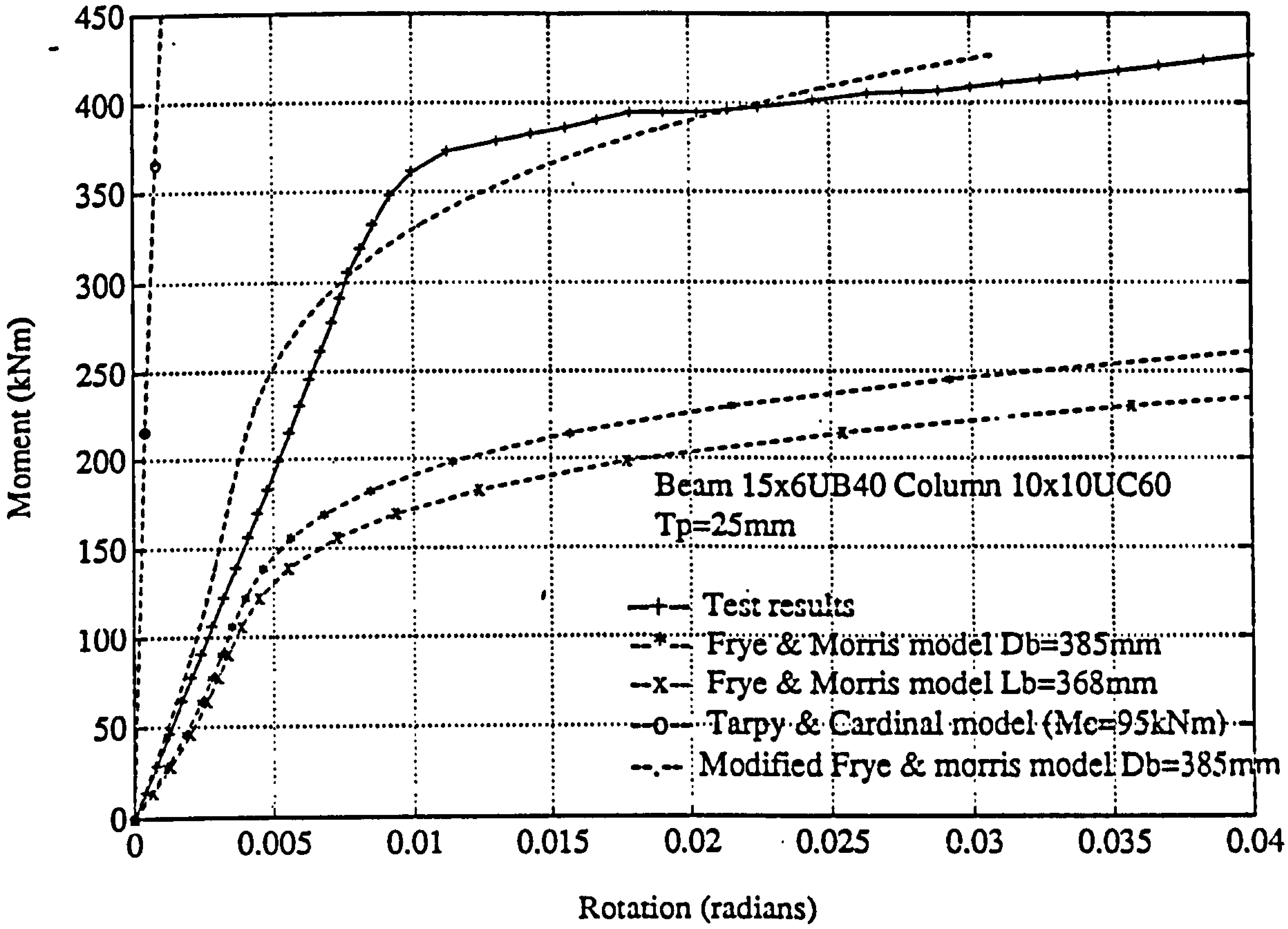


FIG.4-51 Comparison between analytical moment-rotation relationships and Mann test C5 (unstiffened column).

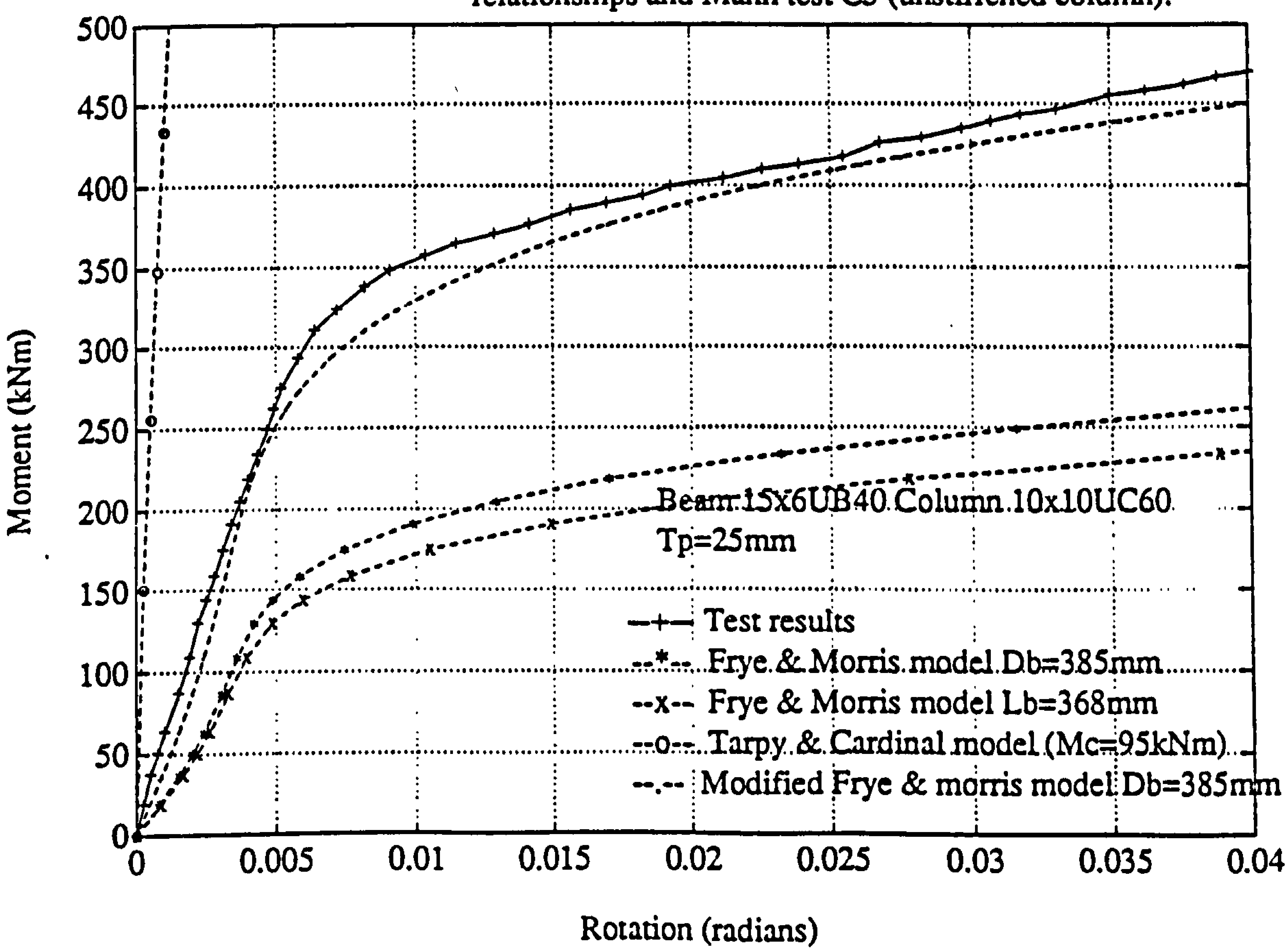


FIG.4-52 Comparison between analytical moment-rotation relationships and Packer test J1 (unstiffened column)

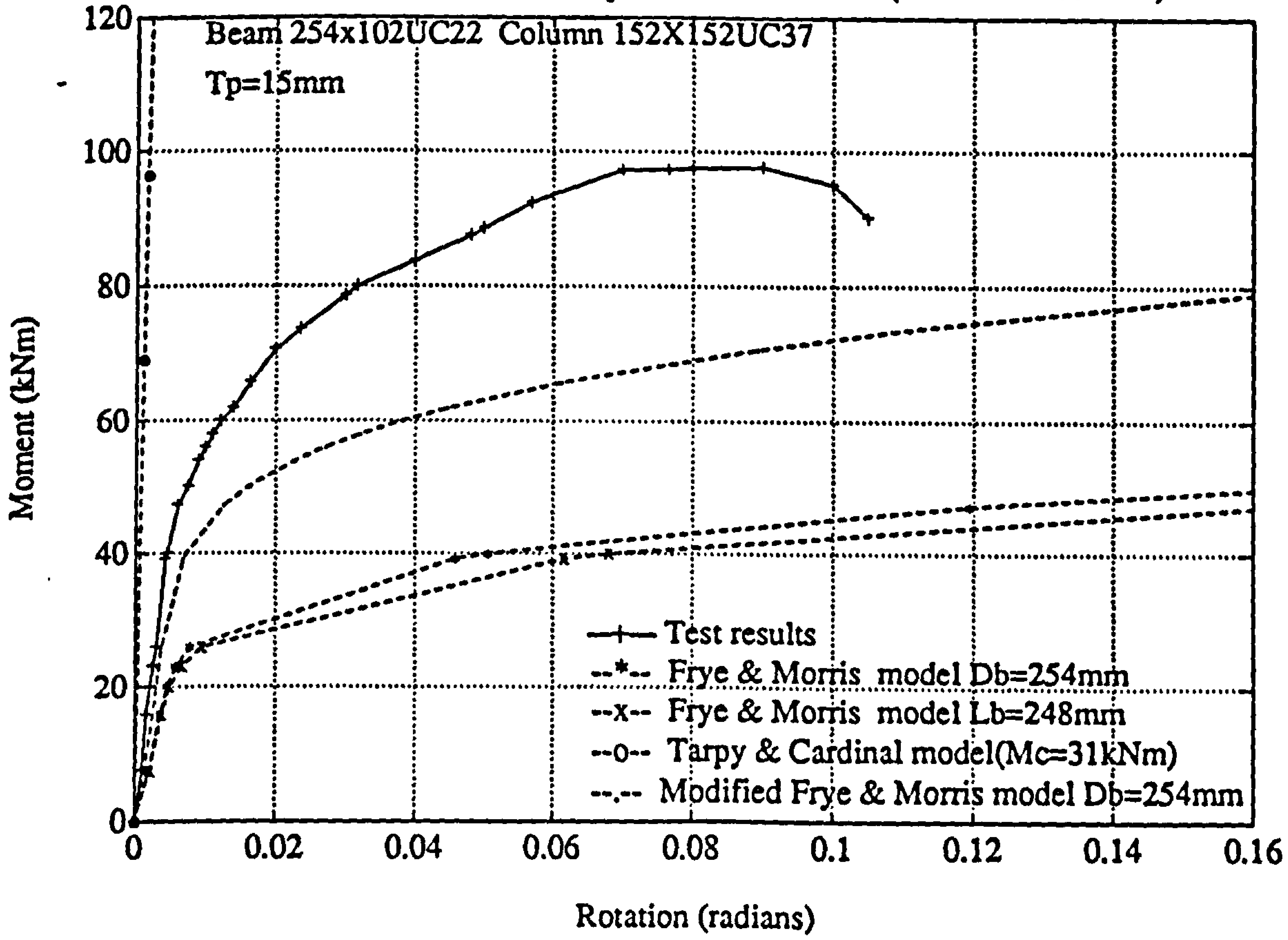


FIG.4-53 Comparison between analytical moment-rotation relationships and Packer test J2 (unstiffened column)

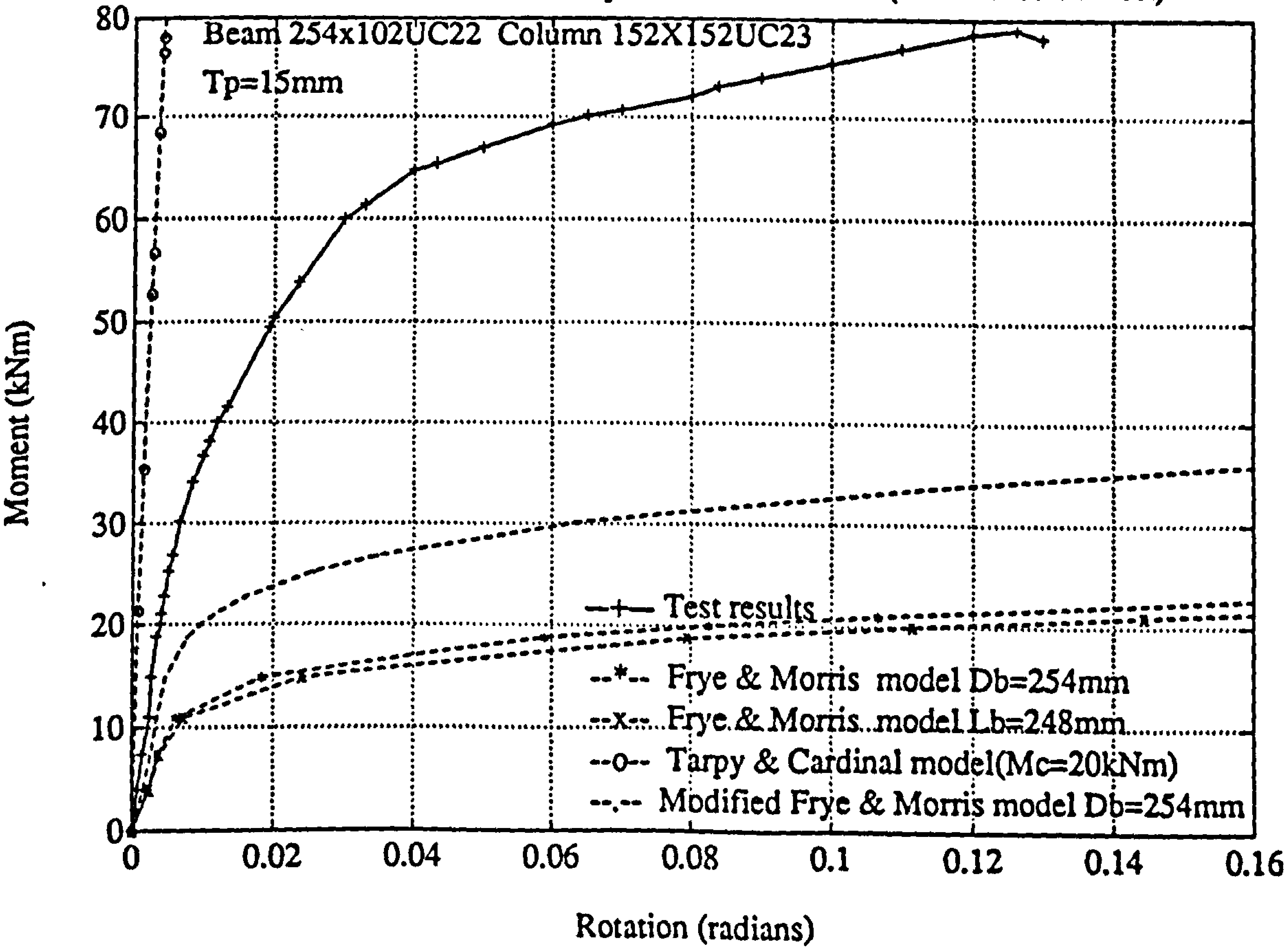


FIG.4-54 Comparison between analytical moment-rotation relationships and Packer test J4 (unstiffened column)

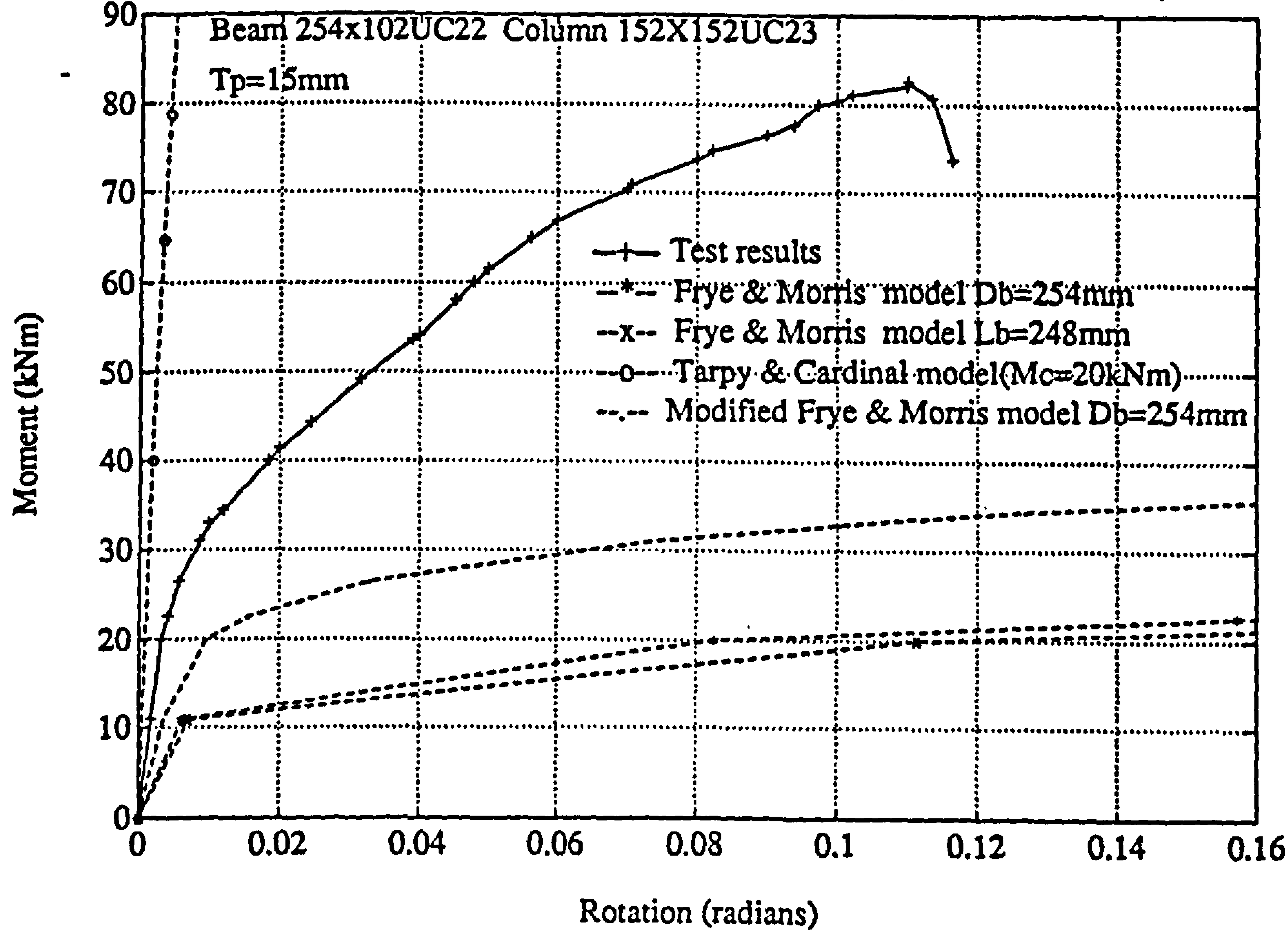


FIG.4-55 Comparison between analytical moment-rotation relationships and Packer test J5 (unstiffened column)

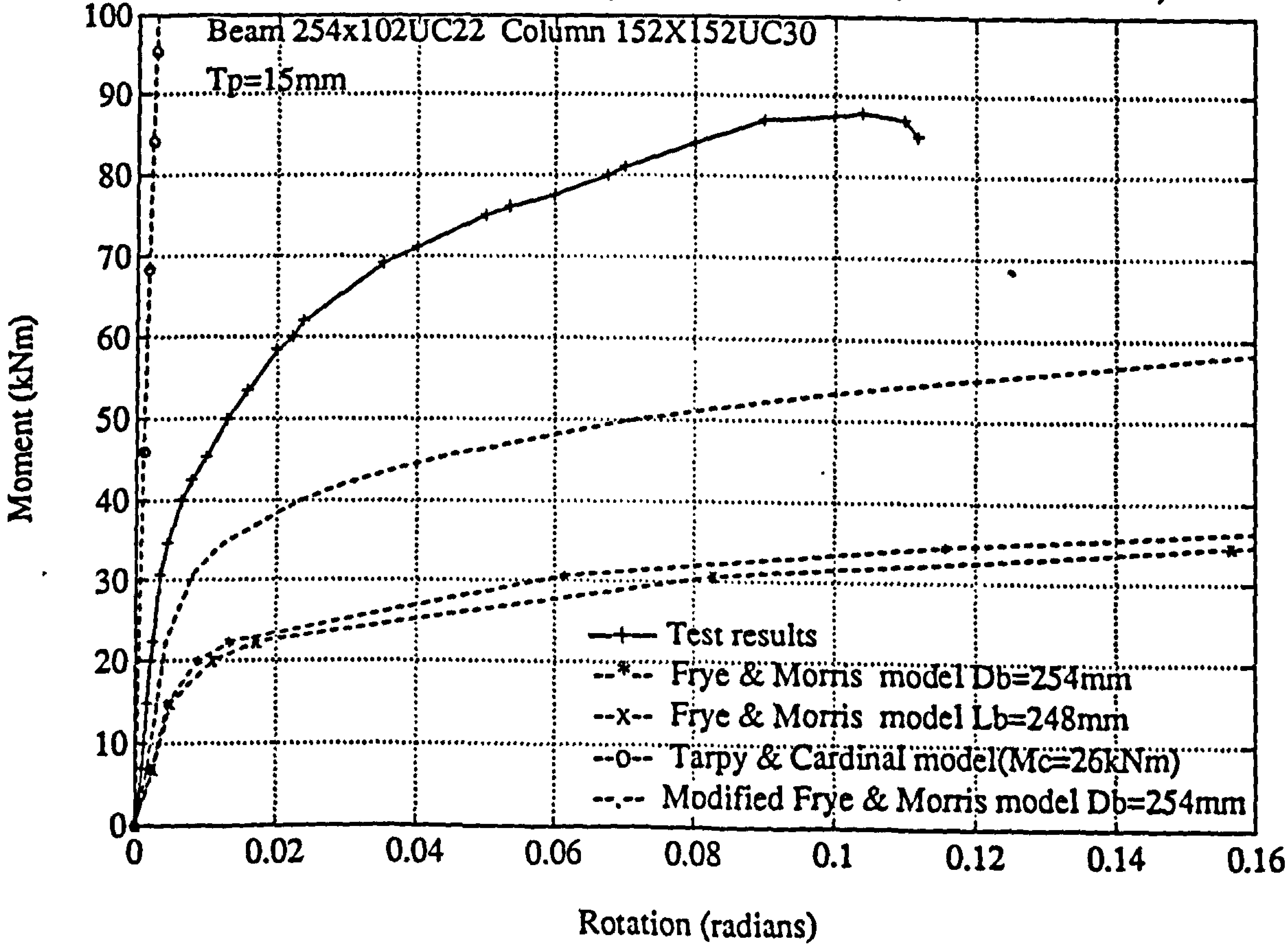


FIG.4-56 Comparison between analytical moment-rotation relationships and Moore & Sims test J3 (unstiffened column)

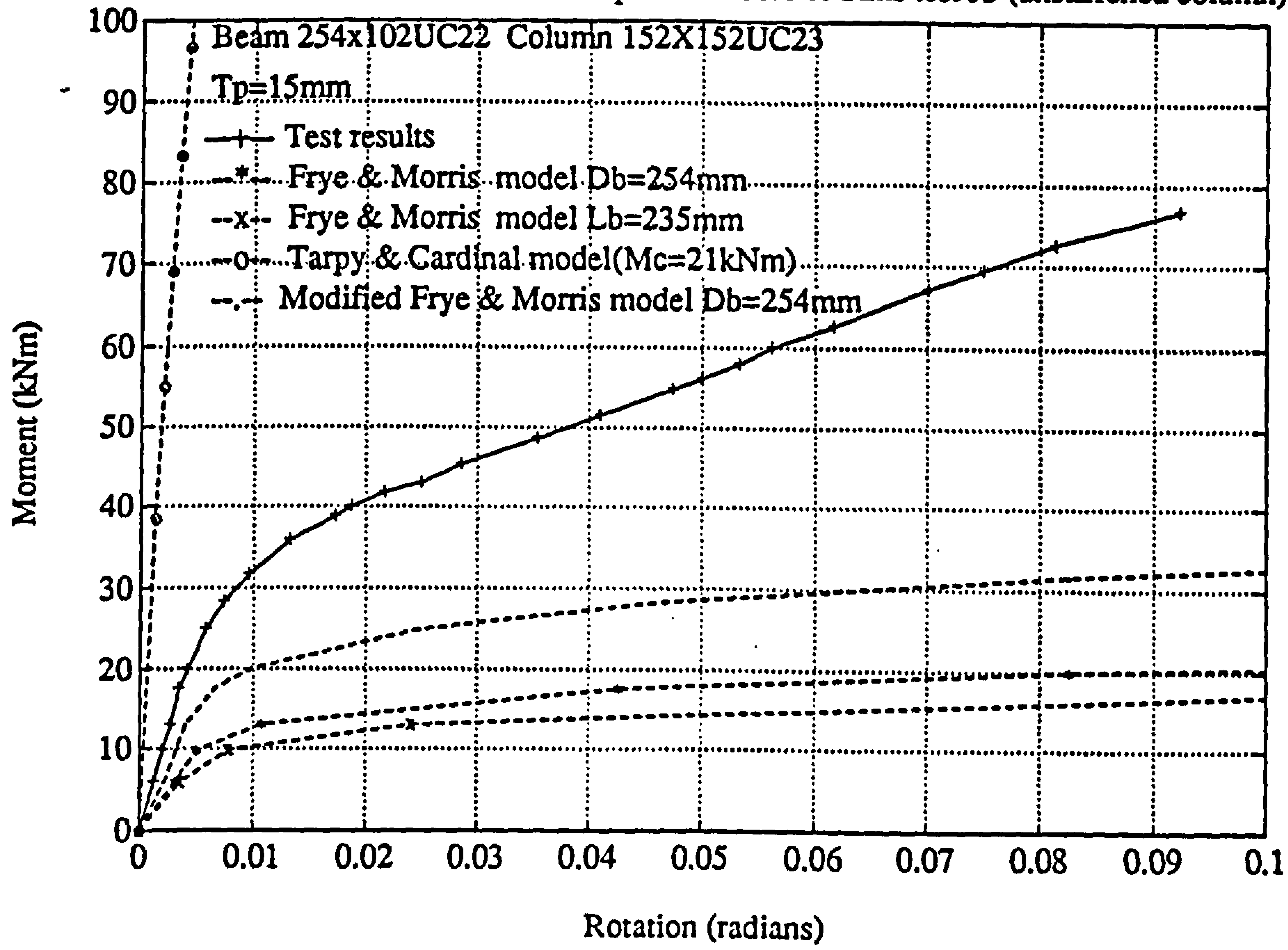


FIG.4-57 Comparison between analytical moment-rotation relationships and Chakrabarti test 1&2 (unstiffened column)

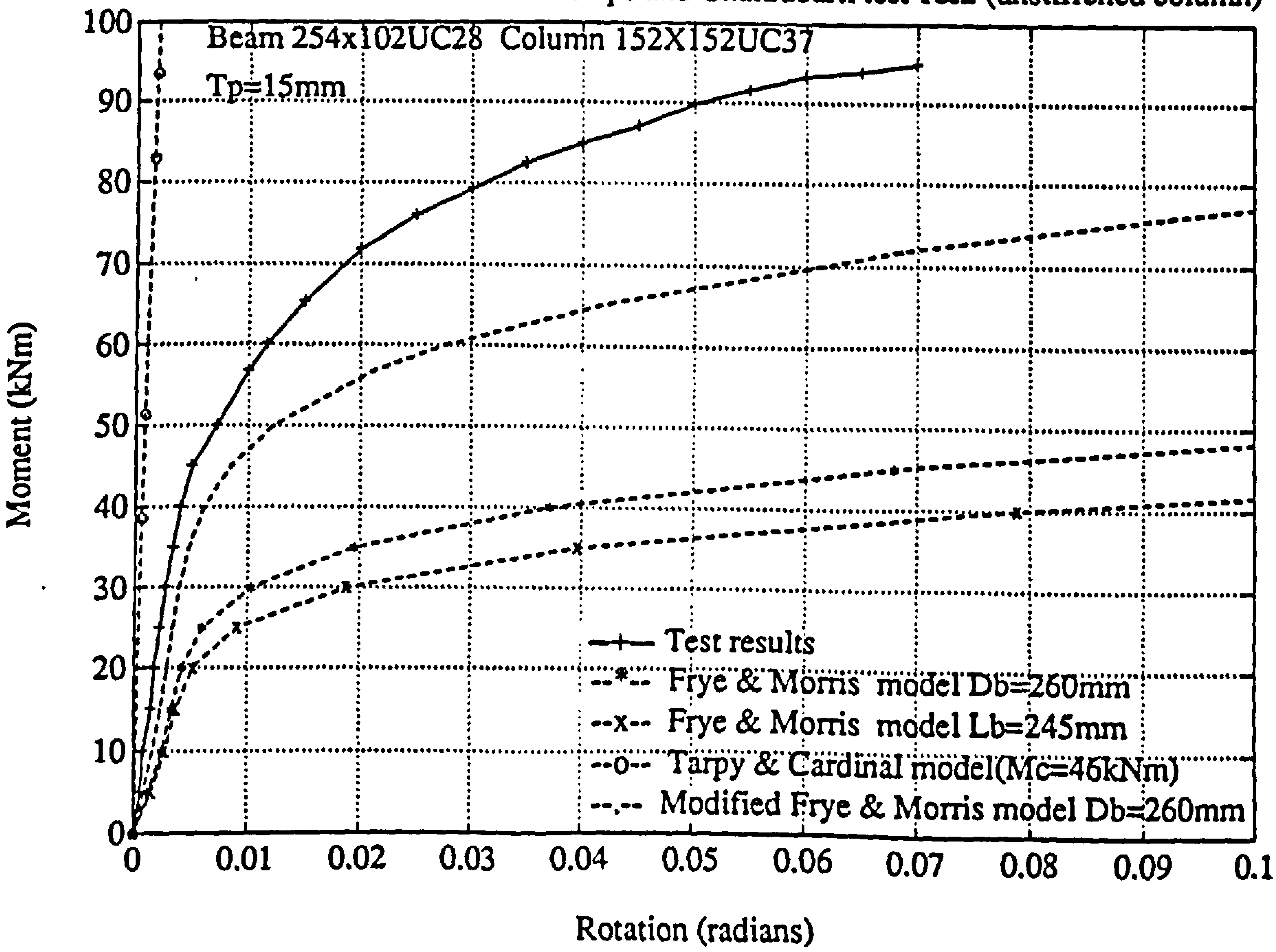


FIG.4-58 Comparison between analytical moment-rotation relationships and Tong test 6 (unstiffened column)

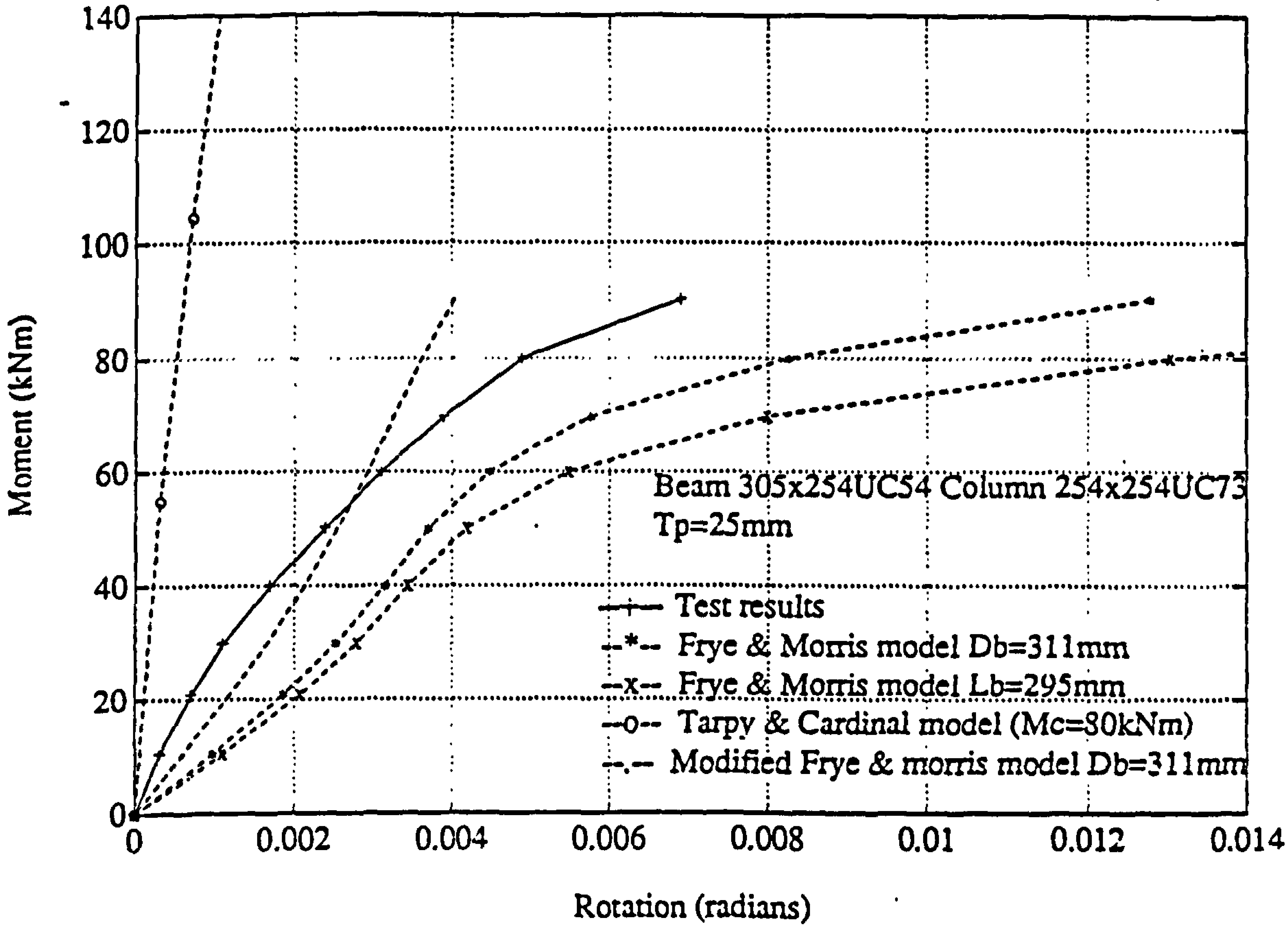
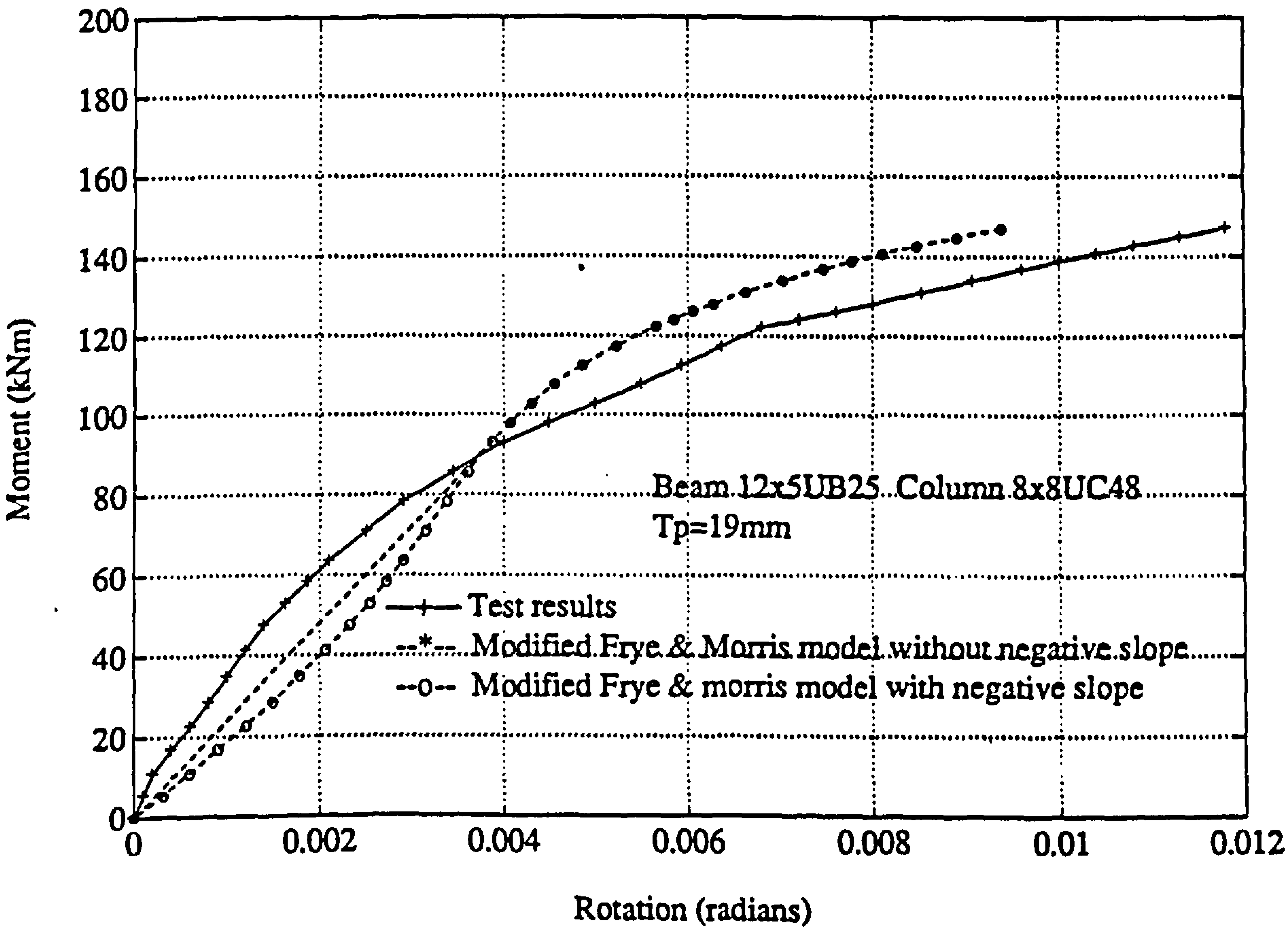


FIG.4-59 Comparison between analytical moment-rotation relationships and Mann test C1 (unstiffened column)



CHAPTER V
ANALYSIS OF SEMI-RIGID STEEL FRAMES

5.1 Introduction

As previously mentioned the principal reason for such interest in beam-to-column connections is that, while design and analysis methods usually are based on assumptions of either fully rigid or completely pinned conditions between the beam and the column, neither condition is in fact ever attained in an actual frame. Based on experimental investigations described in Part A of this thesis, tests on beam-to-column connections indicate that when moment is transferred through the connection, deformations occur in the connection material which result in a relative motion between the beam end and the column which is defined as the connection rotation (see section 1.4). Thus, when moment is applied to the connection, the centreline of the beam does not remain perpendicular to the centreline of the column as presumed in rigid frame analysis; rather an angular distortion occurs due to the flexibility of the connection. The results from connection tests indicate that the moment-rotation relationship is non-linear, even for small moments as illustrated in Figure 1.4.

Having discussed the behaviour and modelling of connections in the preceding chapters, analytical schemes which can be used to predict the response of a flexibly-connected frame need to be investigated. The availability of an increasing volume of connection information has made it possible to include the effects of flexible connections in the analysis of a structural framework.

The objectives of the analysis are principally to produce a research tool capable of analysing plane rectangular steel frames, making a realistic allowance for the semi-rigid behaviour of the beam-to-column connections.

As early as the 1930's both Baker [5.1] and Rathburn [5.2] had applied slope-deflection methods to analyse frames with flexible or semi-rigid connections. They also presented modifications to the stiffness and carry-over factors and the fixed-end moments to permit frames with elastic connections to be analysed by the moment-distribution method. The slope-deflection equations were modified by assuming a linear relationship between the relative rotation of the member at a connection and the bending moment. The method was unsuitable for manual application except for simple frames. For highly redundant structures, therefore, computer methods are necessary. In the same decade, Batho and Rowan [5.1] presented a graphical method, known as the "Beam-Line Method", for predicting the end restraint provided by a connection for which the experimentally obtained moment-rotation relationship was known. Due to calculation time and resulting cost, the method is suitable only for analysing sub-frames and small structures. The beam-line method is discussed in more detail in section 5.2 and its application in section 5.3.

The modification of the slope-deflection and moment-distribution methods used the semi-rigid connection factor Z discussed earlier in section 1.5 and given by equation (1.2). The factor Z represents the inverse of the slope of the assumed straight line portion of the moment-rotation curve shown in Figure 1.9.

The conventional slope-deflection equation for a beam (i-j) of length L and flexural rigidity EI loaded by a uniformly distributed load ω is given by:

$$M_i = \frac{EI}{L} \; (s \; \theta_i + sc \; \theta_j) - \frac{\omega L^2}{12} \qquad (5.1a)$$

$$M_j = \frac{EI}{L} \; (s \; \theta_j + sc \; \theta_i) + \frac{\omega L^2}{12} \qquad (5.1a)$$

where s is known as the stiffness factor or coefficient (because it determines the value of M_i needed to produce a unit rotation at end and i) and c is known as the carry-over factor (because it defines the magnitude of the bending moment transferred, or carried over from end i to end j of the member). For beams with no axial force, s = 4 and c = 0.5. These are the values used in the familiar moment-distribution method for beams.

$$M_i = \frac{EI}{L} \; (4 \; \theta_i + \; 2 \; \theta_j) - \frac{\omega L^2}{12} \qquad (5.2a)$$

$$M_j = \frac{EI}{L} \; (4 \; \theta_j + 2 \; \; \theta_i) + \frac{\omega L^2}{12} \qquad (5.2b)$$

The modified slope-deflection equations for a beam with semi-rigid connections are as follows:

$$M_i = \frac{EI}{L} \cdot \frac{2}{(3\alpha\beta + 2\alpha + 2\beta + 1)} \; [(3\beta + 2)\theta_i + \theta_j] - (3\beta+1) \frac{\omega L^2}{12} \qquad (5.3a)$$

$$M_j = \frac{EI}{L} \cdot \frac{2}{(3\alpha\beta + 2\alpha + 2\beta + 1)} \; [(3\alpha + 2)\theta_j + \theta_i] + (3\alpha+1) \frac{\omega L^2}{12} \qquad (5.3b)$$

where the values of the coefficients α and β in the above equations are defined as:

$$\alpha = \frac{2EI}{L} Z_i \tag{5.4a}$$

and

$$\beta = \frac{2EI}{L} Z_j \tag{5.4b}$$

Z_i and Z_j being the semi-rigid connection factors for end i and j for the beam respectively.

The moment-distribution method of analysis was modified to allow for flexible connections shortly after being first proposed by Hardy Cross. The method was first applied to frames with semi-rigid connections by Baker [5.1] in 1936. The modified moment-distribution factors for a beam of length L and flexural rigidity EI loaded by a uniformly distributed load ω within a frame with semi-rigid connections are given in reference [5.3].

Both the above methods assume a linear moment-rotation relationship which is strictly applicable only for very low values of rotation and becomes increasingly inaccurate as the moment increases, as shown in Figure 1.9. Its magnitude depends on the type of connection.

Beginning in the 1960's, a number of investigators, including Monforton and Wu [5.4], Goble [5.5], and Lightfoot and Le Messurier [5.6] incorporated the effects of connection deformations into a stiffness analysis computer program. However, they assumed linear moment-rotation characteristics based on Lothers [5.7] theoretical investigations. The procedures used in the analysis were generalisations of those that had earlier been added to the slope-deflection and moment-distribution methods. Similar procedures were proposed by Livesley [5.8] and also by Gere and Weaver [5.9]. Correction matrices of the semi-rigid connection restraint were used to modify the conventional stiffness matrix and fixed end force vector. The resulting linear equations are solved as in the normal stiffness method.

As the connections exhibit a non-linear behaviour over almost the entire range of loading, the procedure would give misleading and unacceptable results if applied to these connections. A bilinear relationship was used by Romstad and Subramaniam [5.10] to investigate the analysis of frames with partial connection rigidity and also by Lionberger and Weaver [5.11], to study the dynamic response of frames with flexible connections. A trilinear model was presented by Moncarz and Gerstle [5.12].

In 1975, Frye and Morris [5.13] presented an iterative analysis procedure for planar, rectangular steel frames involving repeated cycles of linear analysis incorporating the non-linear connection effects of any of seven beam-to-column connection types. After each cycle, the flexibilities are modified and the new connection flexibilities used to change the member stiffness matrices and the member fixed-end-forces for the next analysis. The procedure continues until the rotation and moment calculated for each connection, in the linear analysis, satisfy the equation of the non-linear moment-rotation curve for the connection.

In 1984, Ang and Morris [5.14] generalised the Frye and Morris [5.13] procedure to permit the analysis of three-dimensional rectangular frames with non-linear flexible connections. They considered the non-linear M-Φ behaviour which they modelled using the Ramberg-Osgood function. The P-δ effects were also included in the analysis procedure. Both the Frye and Morris and Ang and Morris procedures assumed proportional loading. Thus, they did not permit unloading of any connection.

Various approaches are currently available for the analysis of flexibly connected frames. Different types and degree of non-linearity are considered in the analysis. The non-linearities in a flexibly connected frame consist of three types: (i) the non-linear M-Φ relationship of the connection; (ii) the geometrical non-linearity of the member referred usually as P-δ effects; and (iii) the material non-linearity or yielding in the members of the frame. Thus, depending on the types of non-linearity and the degree of accuracy required, three different analysis techniques can be used. These are:

i) Linear elastic analysis.

ii) Non-linear elastic analysis.

iii) Inelastic analysis.

The scope of the present work is limited to a non-linear elastic analysis of flexibly connected frames which also included the non-linearity M-Φ relationship of the connections. Before proceeding to the analytical techniques and methods of analysis, the beam-line method is first discussed with its application in simple sub-frames analysis.

5.2 Beam-line method

The method was proposed by Batho and Rowan [5.1] and later developed by Batho [5.1] in order that design engineers could predict the performance of beam-column connections when incorporated in a structural frame.

The fundamental expression of the beam-line concept can be derived from the slope-deflection equation. Equation (5.5) gives the end rotation Φ, of a beam of span L, with flexural stiffness EI, equal end moments M, and loaded with a uniformly distributed load ω.

$$\Phi = \frac{\omega L^3}{24EI} - \frac{ML}{2EI} \qquad (5.5)$$

For a fixed ended beam, the end rotation is zero and from the above expression, the corresponding end moment is $\omega L^2/12$. For a simply supported beam the end moment M_e is zero and the corresponding end rotation is $\omega L^3/24EI$.

For a given value of ω, the two end points of the beam-line are known and hence the conventional beam-line is obtained as shown in Figure 5.1. Also shown in Figure 5.1 is the moment-rotation curve of a typical connection. The beam-line and the moment-rotation curve intersect at a point, L, where the value of M_ℓ for moment and Φ_ℓ for angle of rotation represent the end restraint conditions that would exist at the end of such a member with the connection as described by the moment-rotation curve. The advantage of this method is that it uses the actual M-Φ relationship and so finds a more accurate value of end restraint without assuming linear moment-rotation behaviour. However, the method requires experimental or analytical moment-rotation data to be available for every connection analysed.

5.2.1 The yield beam-line

When comparing the behaviour of two semi-rigid connections a realistic comparison is attained by considering the interaction of each connection with the beam at the condition of yield in a semi-rigid structural frame. The stiffer the connection, the greater the uniform load the beam will be able to sustain before the occurrence of first yield of the extreme fibres at some location in the span of the beam. Sommer [5.15] developed a beam-line for the condition when the yield moment is achieved either at the ends or at mid span of the beam, which he named the yield beam-line. This yield beam-line is shown in Figure 5.2.

As with the conventional beam-line, the two end points, M_y and Φ_o, of a yield beam-line can be readily calculated.

For a fixed ended beam,

$$M_e = M_y = \omega_e L^2/12 \text{ and } \Phi = 0$$

where:

M_y is the beam yield moment,

ω_e is the uniform load applied to a particular fixed ended beam to cause the beam to yield at the support.

For a simply supported beam,

$$\Phi = \Phi_o = \omega_o L^3/24EI, \; M = 0$$

and also $M_s = M_y = \omega_o L^2/8$

where:

M_s is the mid span moment;

ω_o is the uniform load applied to a simply supported beam to cause yield at mid span.

Assuming that M_y is constant for any length of beam, it follows that:

$$M_s = M_e = M_y$$

Thus,

$$\omega_e L^2/12 = \omega_o L^2/8$$

and therefore

$$\omega_e = 3/2 \; \omega_o.$$

The fact that ω_e is not equal to ω_o indicates that the total uniform load applied to a beam to cause first yield is not a constant and varies along the yield beam-line.

Sommer [5.15] has also shown that the yield beam-line representing the attainment of first yield at some point in the beam must consist of two phases.

5.2.2 The two-phase beam-line

In the two phase beam-line, one phase represents yielding of the supports and the other phase represents yielding at mid span. Considering first yielding at mid span, equation (5.5) can still be applied.

$$\Phi = \frac{\omega L^3}{24EI} - \frac{ML}{2EI} \tag{5.5}$$

The total static mid span moment is equal to $\omega L^2/8$ and this must equal to $M_S + M$. If yield occurs at mid span,

$$M_S = M_y$$

and

$$\omega L^2/8 = M_y + M \text{ or } \omega = 8(M_y + M)/L^2$$

Substituting for ω in equation (5.5) leads to:

$$\Phi = \frac{M_y L}{3EI} - \frac{ML}{6EI} \tag{5.6}$$

Equation (5.6) is a linear relationship in M and Φ, representing a beam with varying ω and for which yield occurs at mid span. For a simply supported beam, $M = 0$ and $\Phi = \Phi_o = M_y L/3EI$.

If the connection is such that yielding occurs at mid span and at the supports simultaneously, substitution of $M = M_y$ in equation (5.6) results in,

$$\Phi = M_y L/6EI = \Phi_o/2$$

Thus the first phase for yielding at mid-span is represented by equation (5.6),

$$\Phi = M_y L/3EI - ML/6EI \text{ for } \Phi_o/2 \leqslant \Phi \leqslant \Phi_o$$

The two-phase beam-line is illustrated in Figure 5.3. The various beam-lines which have been developed are compared in Figure 5.4.

For a given beam, $M_y = F_y Z$

where

F_y is the yield strength of the steel,

Z is the section modulus of the given beam.

Substituting for M_y in the expression for Φ_0 and realising that the elastic section modulus Z is given by:

$$Z = \frac{I}{D_b/2}$$

where D_b is the depth of the beam, one obtains

$$\Phi_o = (2F_y/3E)\ (L/D_b)$$

Therefore for a given span to depth ratio, material and section properties of the beam, it is possible to determine Φ_0 and M_y and hence the two-phase beam-line for each connection.

Here the beam-line has been defined for a uniformly distributed load but this can be done in the same for any type of loading.

5.3 Analysis of semi-rigid sub-frame using beam-line method

The earlier part of this chapter is a review of previous work. However, semi-rigidly connected sub-frames are analysed by the beam line method by the present author in this section.

Figure 5.5 shows a series of sub-frames where the beam has various end conditions. The beam is of span L, with flexural stiffness EI and loaded with a uniformly distributed load ω. In Figure 5.5a, the beam moments do not depend on properties of columns and the order of design is beams first then columns. For rigid connections (Figure 5.5b), the beam moments depend on the relative stiffness of columns to beams. Column sizes must be guessed first, followed by design of the beams; there is possibly the need to iterate. However for semi-rigid connections (Figure 5.5c) the beam moments depend on properties of connections, on the relative stiffnesses of columns to beams and connections to beams.

For a multi-bay braced frame, for equal loading and equal spans the middle column will not rotate (see Figure 5.5d) and therefore each beam can be treated as shown in Figure 5.5e. However, the maximum sagging moment in one beam will arise when that span is fully loaded, and the adjacent span carries only minimum dead load, as shown in the sub-frame of Figure 5.5f. In this case the moments in the beam being designed depend on all the parameters listed at the end of the previous paragraph.

Clearly it would be desirable if a beam with semi-rigid connections could be analysed without reference to the adjacent members, as is possible in simple design. This may be possible provided that the stiffness of the connections is low in relation to that of the surrounding members.

If the beam can be analysed without reference to the column then the behaviour of the beam can be determined by finding where the M-Φ curve for the joint intersects the beam line as shown in Figure 5.1.

It is expected that for high column to beam stiffness ratio and less stiff connections, the influence of column flexibility will be negligible. For low column to beam stiffness ratio and more stiff connections, column flexibility will significantly affect the beam moments. For intermediate values of column to beam stiffness ratio a limiting column to beam stiffness ratio should be determined such that neglect of column flexibility will not affect beam design moments significantly.

In this study only the header plate connection is considered with the assumption of pins at far ends of columns first then fixed ends. The present author is examining one bay sub-frames as an extreme case of a more general multi-bay frame with unequal spans and/or loadings i.e. a multi-bay frame where the adjacent beams do not apply any hogging moments to the end of the beam being designed. Therefore the worst case for sagging moment is examined by looking at a single bay frame.

The beam-to-column connection shown in Figure 5.6a represents a flexible end-plate connection used in a frame designed by Pask [5.16]. This connection represents a particular numerical example for the present study.

Using equations (2.18) and (2.19), the representation of joint behaviour is given by:

$$\Phi = 3.777 \times 10^{-7}\, M + 2.518 \times 10^{-16}\, M^3 + 5.345 \times 10^{-24}\, M^5 \qquad (5.7)$$

where Φ is the connection rotation in radians and M is the end-moment in kNcm. The beam behaviour for a uniformly distributed load ω = 100 kN/m and span L = 7.50 m, is as follows:

For fully fixed ends beams $M_e = \dfrac{\omega L^2}{12} = 468$ kNm.

For a pin ended beam $\Phi_o = \dfrac{\omega L^3}{24EI} = 15.122 \times 10^{-3}$ radians.

The beam-line and restraint-line intersect at a point, P, where the values of moment is 148 kNm and angle of rotation 10.277×10^{-3} radians (see Figure 5.7, intersection of curve (a) with the beam line(e)). Therefore by taking the end restraint into consideration;

beam end moment $M_e = 150$ kNm , and

mid span moment $M_s = M_f - M_e = 553$ kNm.

where M_f is the free moment.

Column flexibilities are ignored in the above particular numerical example (i.e. beam analysed without reference to the column but properties of connections are taken into consideration). This particular connection will be used in sub-frames SA1 and SB1.

5.3.1 Sub-frame SA1

Consider the sub-frame shown in Figure 5.6b with the joint behaviour as given previously by equation (5.7).

a) Analysis of sub-frame SA1 using beam-line method:

For the sub-frame SA1 shown in Figure 5.6b, the slope-deflection equations provide the following analysis if the flexibility of columns is taken into consideration:

$$M_{AB} = \frac{2EI}{L}(2\theta_A + \theta_B) = 0$$

Thus, $\theta_A = -\theta_B/2$

Also $M_{BA} = \dfrac{2EI}{L}(\theta_A + 2\theta_B)$

Substituting θ_A by $-\theta_B/2$ in the M_{BA} expression gives:

$$M_{BA} = \frac{3EL}{L}\theta_B$$

or

$$\theta_B = \left[\frac{M_{BA} \cdot L}{3EI}\right]_{col}$$

If M is end moment from the beam, then for columns of equal stiffness M_{BA} = M/2, hence

$$\theta_B = \left[\frac{ML}{6EI} \right]_{col} \tag{5.8}$$

Using the properties of the sub-frame in Figure 5.6b, this gives θ_B = 7.682 x 10^{-8} M. Adding this to the connection rotation in equation (5.7), the total rotation (θ + Φ) is:

$$\theta{+}\Phi = 4.545 + 10^{-7}\ M + 2.518 \times 10^{-16}\ M^3 + 5.345 + 10^{-24}\ M^5 \tag{5.9}$$

This revised analysis including column flexbility results in curve (b) in Figure 5.7. From the intersection of this with the beam-line(e) it is found that:

beam end moment M_e = 144 kNm, and

mid span moment M_s = 559 kNm.

Therefore the increase in mid span moment due to column flexibility is 553/559 = 0.989 i.e. about 1%

b) Using algebraic analysis

Although the moment-rotation relationships are generally non-linear, a linear approximation will be assumed for the purpose of simplifying the analysis at this stage. Thus the connection moment-rotation relationship becomes:

$$\Phi = C_1\ K\ M \tag{5.10}$$

In other words, the terms $M^3/10^{16}$ and $M^5/10^{24}$ in equ. (5.9) are ignored and the joint rotation is given by equ. (5.10). The column rotation is defined by equation (5.8) as:

$$\Phi_{col} = \left[\frac{L}{6EI} \right]_{col} . M$$

The total connection rotation can be written as:

$$\Phi_{Total} = \Phi_{Joint} + \Phi_{column}$$

or

$$\Phi_T = \left[C_1 K + \left(\frac{L}{6EI}\right)_{col}\right] M \tag{5.11}$$

The beam line equation is as follows:

$$\Phi = \left(\frac{WL^2}{24EI}\right)_{beam} \cdot \left(1 - \frac{M}{M^F_e}\right) \tag{5.12}$$

In equ. (5.12) W represents the total load (i.e. in kN) and $M^F{}_e$ is the beam end moment corresponding to a fixed ends.

The intersection of the beam line (equ. (5.12)) and the restraint-line (equ. (5.11)) leads to the following equation if the load is uniformly distributed:

$$\left[C_1 K + \left(\frac{L}{6EI}\right)_{col}\right] M = W \left(\frac{L^2}{24EI}\right)_{beam} \cdot \left(1 - \frac{M}{\left(\frac{WL}{12}\right)_{beam}}\right)$$

Rearranging the above equation gives:

$$M \left\{C_1 K + \left(\frac{L}{6EI}\right)_{col} + \left(\frac{L}{2EI}\right)_{beam}\right\} = W \left(\frac{L^2}{24EI}\right)_{beam}$$

Therefore, the beam end moment is given by:

$$M_e = W \left(\frac{L^2}{24EI}\right)_{beam} \cdot \left\{C_1 K + \left(\frac{L}{6EI}\right)_{col} + \left(\frac{L}{2EI}\right)_{beam}\right\}^{-1} \tag{5.13}$$

and the mid span moment becomes:

$$M_s = \left[\frac{WL}{8}\right]_{beam} - M_e$$

or

$$M_s = \left[\frac{WL}{8}\right]_{beam} - W\left[\frac{L^2}{24EI}\right]_{beam} \cdot \left\{C_1K + \left[\frac{L}{6EI}\right]_{col} + \left[\frac{L}{2EI}\right]_{beam}\right\}^{-1} \qquad (5.14)$$

If column flexibility is ignored, i.e. column very stiff:

$$\left[\frac{L}{6EI}\right]_{col} \simeq 0$$

then, equation (5.14) leads to:

$$M_s = \left[\frac{WL}{8}\right]_{beam} - W\left[\frac{L^2}{24EI}\right]_{beam} \cdot \frac{1}{C_1K + \left[\frac{L}{2EI}\right]_{beam}} \qquad (5.15)$$

If α' is defined as the ratio of mid span moment ignoring column flexibility to mid span moment including column flexibility, then:

$$\alpha' = \frac{\left[\frac{WL}{8}\right]_{beam} - \left[\frac{WL^2}{24EI}\right]_{beam} \cdot \frac{1}{C_1K + \left[\frac{L}{2EI}\right]_{beam}}}{\left[\frac{WL}{8}\right]_{beam} - \left[\frac{WL^2}{24EI}\right]_{beam} \cdot \frac{1}{C_1K + \left[\frac{L}{6EI}\right]_{col} + \left[\frac{L}{2EI}\right]_{beam}}} \qquad (5.16)$$

Let

$$a = \left[\frac{L}{EI}\right]_{beam}$$

$$x = (C_1 K)_{con} \text{ , and}$$

$$y = \left[\frac{L}{6EI} \right]_{col}$$

Dividing the numerator and the denominator of equ. (5.16) by $(L/8)_{beam}$ and substituting the flexibility terms $(L/EI)_{beam}$, $(C_1 K)_{con}$ and $(L/6EI)_{col}$ respectively by a, x and y; one obtains:

$$\alpha' = \frac{1 - \dfrac{a}{3} \cdot \dfrac{1}{x + a/2}}{1 - \dfrac{a}{3} \cdot \dfrac{1}{x + y + a/2}} \tag{5.17a}$$

or

$$\alpha' = \frac{1 - \dfrac{1}{3(x/a + 1/2)}}{1 - \dfrac{1}{3(x/a + y/a + 1/2)}} \tag{5.17b}$$

Before plotting the contour x/a versus y/a for a fixed α', it would be better to work in terms of $(L/EI)_{col}$ than $(L/6EI)_{col}$ term. Let now $z = (L/EI)_{col}$ and substituting y in equ. (5.17b) by z/6, it follows:

$$\alpha' = \frac{1 - \dfrac{1}{3(x/a + 1/2)}}{1 - \dfrac{1}{3(x/a + z/6a + 1/2)}} \tag{5.18}$$

To plot equ. (5.18) for a particular value of α', choose a value of x/a and solve equations (5.18) for z/a, i.e.

$$1 - \frac{1}{3\ x/a + 1.5} = \alpha' - \frac{\alpha'}{3\ x/a + 0.5\ z/a + 1.5}$$

This leads to:

$$z/a = \frac{2\alpha'}{\alpha' - 1 + \dfrac{1}{3\ x/a + 1.5}} - 2\ (3\ x/a + 1.5) \qquad (5.19)$$

Let $\beta' = 3\ x/a + 1.5$ and substitute its value into equ. (5.19); if follows

$$z/a = 2\left\{\frac{\alpha'}{\alpha' - 1 + 1/\beta'} - \beta'\right\} \qquad (5.20a)$$

or

$$z/a = \frac{2\alpha'\ \beta'}{1 + (\alpha' - 1)\ \beta'} - 2\beta' \qquad (5.20b)$$

For the header plate connection shown in Figure 5.6a:

$\Phi = C_1\ K\ M = 3.777 \times 10^{-7}\ M$, ($\Phi$ in radians and M in kNcm)

$a = (L/EI)_{beam} = 6.4521 \times 10^{-7}$, (in 1/kNcm)

$x = C_1 K = 3.777 \times 10^{-7}$,

$x/a = 0.585$,

$y = (L/6EI)_{col} = 7.6825 \times 10^{-8}$

$y/a = 0.119$,

$z/a = 0.714$, and

$\beta' = 3.255$

Substituting the above numerical values into equ. (5.18), the ratio α' given by the algebraic analysis is 0.958, i.e. an error of 4%. This result is confirmed by a graphical plot of the beam-line method in conjunction with the linear M-Φ relationship (see Figure 5.7):

i) ignoring column flexibility (hence $\Phi = 3.777 \times 10^{-7}$ M from equ. (5.7)) the intersection of curves (e) with (c) gives:

$M_e = 217$ kNm.

Therefore $M_s = 486$ kNm;

ii) including column flexibility (hence $\Phi = 4.545 \times 10^{-7}$ M, from equ. (5.9)) the intersection of line (d) with the beam-line (e) in Figure 5.7 gives:

$M_e = 194.5$ kNm

Therefore $M_s = 508.6$ kNm

The ratio of mid span moment by ignoring column flexibility to mid span moment including column flexibility, α', is:

$\alpha' = 486/508.6 = 0.956$

This value is very close to the value given by the algebraic analysis (i.e. $\alpha' = 0.958$).

Conclusions to be drawn from the above case are:

1. i) Ignoring column flexibility:

- with non linear M-Φ relationship

  $M_s = 553$ kNm

- with linear M-Φ relationship,

  $M_s = 486$ kNm

Therefore the error in mid span moment by assuming a linear M-Φ relationship is ((553-486)/553) x 100% (i.e. the assumption of a linear

M-Φ relationship underestimates the true value of mid span moment by about 12%).

ii) Including column flexibility,

- with non-linear M-Φ relationship

$M_S = 559$ kNm

- with linear M-Φ relationship.

$M_S = 508.6$ kNm

Therefore the error in mid span moment by assuming a linear M-Φ relationship is ((559-508.6)/559) x 100% (i.e. the linear M-Φ relationship assumption underestimates the true value of mid span moment by about 9%).

2. Considering the effect of column flexibility on the mid span moment:

i) In a non linear M-Φ relationship

error = 553/559 = 0.989 i.e. about 1%

ii) In a linear M-Φ relationship

error = 486/508.6 = 0.956 i.e. about 4.5%

c) **Variations of column to beam flexibility ratio and connection to beam flexibility ratio for a fixed allowable error:**

Let the allowable error in beam moments from neglect of column flexibility to be 10%, in other words $\alpha' = 0.90$. The required limiting column to beam flexibility ratio and connection to beam flexibility ratio will now be determined. Substituting α' by its value 0.9 in equation (5.20b), it follows:

$$z/a = \frac{1.8\beta'}{1 - 0.1\beta'} - 2\beta' \tag{5.21}$$

Values of column to beam flexibility ratio, z/a, are given in Table 5.1 for connection to beam flexibility ratio, x/a, values ranging from 0.10 to 0.50 for α' = 0.90. Values are also given for α' = 0.95.

The results shown in Table 5.1 are plotted in Figure 5.8.

i) Interpretation of the results:

If x/a is low, the connection is not flexible relative to the beam and the error in ignoring column flexibility will only be small if column is very stiff. Increasing connection flexibility permits a less stiff column for the same error, i.e. limiting z/a increases.

For the particular example used previously where x/a = 0.585 and z/a = 0.714, a location of the co-ordinates (0.585, 0.714) in Figure 5.8 indicates the error is about 4.5% which is already justified by the beam-line method.

ii) In terms of stiffnesses:

Because designers are more used to work with stiffness terms rather than flexibility terms, the above analysis needs to be converted into terms of stiffnesses.

Column stiffness = $(EI/L)_{col}$ = 1/z

Connection stiffness = $1/C_1 K$ = 1/x

Beam stiffness = $(EI/L)_{beam}$ = 1/a

The plots need to be presented in terms of:

$$\frac{\text{connection stiffness}}{\text{beam stiffness}} = \frac{1/x}{1/a} = a/x$$

$$\frac{\text{connection stiffness}}{\text{beam stiffness}} = \frac{1/z}{1/a} = a/z$$

hence from equ. (5.20a)

$$a/z = \frac{1}{2\left\{\dfrac{\alpha'}{\alpha' - 1 + 1/\beta'} - \beta'\right\}}$$

where $\beta' = 3\ x/a + 1.5$

or

$$\beta' = \frac{3}{a/x} + 1.5$$

Table 5.2 summarizes the analysis results in terms of stiffnesses for $\alpha' = 0.90$ and $\alpha' = 0.95$. The results are plotted in Figure 5.9.

Thus Figures 5.8 and 5.9 show the influence of column flexibility on the beam's mid span moment assuming a linear M-Φ relationship for header plate connections in the general case of sub-frames of similar shape to SA1 (i.e. columns pinned at the far end of the sub-frame). Note though that for a beam with a high degree of end restraint, the end moment may govern the design of the beam. Such end restraint is unlikely in practice though with header plate connections.

5.3.2 Sub-frame SB1

Consider the sub-frame shown in Figure 5.6c with the joint behaviour as given previously by equation (5.7). In Figure 5.6c M is the bending moment from the connected beam (i.e. end-moment of the beam).

As the ends of the upper and lower column are now assumed fixed, then

$$\theta_A = \theta_C = 0.$$

The slope deflection equations give:

$$M_{AB} = \frac{EI}{L}(4\theta_A + 2\theta_B) = \frac{2EI}{L}\theta_B$$

$$M_{BA} = \frac{EI}{L}(4\theta_B + 2\theta_A) = \frac{4EI}{L}\theta_B$$

$$M_{BA} = \frac{M}{2} = \left[\frac{4EI}{L}\right]_{col}\theta_B$$

$$\text{hence } \Phi_c = \left[\frac{L}{8EI}\right]_{col} M \qquad (5.22)$$

where Φ_c is the rotation due to column flexibility. Therefore the total rotation of the connection region is given by:

$$\Phi_T = \Phi_{joint} + \Phi_{column}$$

where Φ_{joint} is given by equation (5.7).

Thus for the sections shown in Figure 5.6a,

$$\Phi_T = 4.353 \times 10^{-7} M + 2.518 \times 10^{-16} M^3 + 5.345 \times 10^{-24} M^5 \qquad (5.23)$$

Using the beam line method shown in Figure 5.10, the beam end moment is M_e= 146 kNm, and the mid span moment is M_s = 557 kNm for the particular numerical example where the uniformly distributed load is 100 kN/m and the beam span is 7.50m.

The error in mid span moment by ignoring column flexibility is equal to:

((557-553)/557) x 100% = 0.72% (about 1%)

The algebraic analysis is carried out exactly in the same way as for sub-frame SAl, ignoring the terms $(KM)^3$ and $(KM)^5$; the beam end moment is given by the following equation:

$$M_e = W \left[\frac{L^2}{24EI}\right]_{beam} \left\{ C_1 K + \left[\frac{L}{8EI}\right]_{col} + \left[\frac{L}{2EI}\right]_{beam} \right\}^{-1} \qquad (5.24)$$

The corresponding mid span moment is as follows:

$$M_s = \left[\frac{wL}{8}\right]_{beam} - W \left[\frac{L^2}{24EI}\right]_{beam} \left\{ C_1 K + \left[\frac{L}{8EI}\right]_{col} + \left[\frac{L}{2EI}\right]_{beam} \right\}^{-1} \qquad (5.25)$$

Obviously the only difference between equ. (5.13) and (5.14) and equ. (5.24) and (5.25) is the term involving column flexibility. Therefore the factor y is now defined as $(L/8EI)_{col}$. The analysis is continued in the same way as previously and leads to:

$$\alpha' = \frac{1 - \dfrac{1}{3\ x/a + 1.5}}{1 - \dfrac{1}{3\ x/a + 0.375\ z/a + 1.5}} \qquad (5.26)$$

where

$$a = \left[\frac{L}{EI}\right]_{beam}$$

$x = C_1 K$, and

$z = (L/EI)_{col} = 8y$

Solving equ. (5.26) for a particular values of α' and x/a, it results:

$$z/a = \left[\frac{\alpha'}{\alpha' - 1 + 1/\beta'} - \beta'\right] .\ 8/3 \qquad (5.27)$$

Substituting the numerical values for the header plate connection and the sub-frame (Figure 5.6) into equ. (5.26), the error α' given by the algebraic analysis is 0.967 for this particular example. Using the beam-line method and assuming a linear M-Φ relationship (see Figure 5.10), the following results are obtained:

i) ignoring column flexibility,

$\Phi = 3.777 \times 10^{-7}$ M

$M_e = 217$ kNm

$M_s = 486$ kNm

ii) including column flexibility,

$\Phi = 4.353 \times 10^{-7}$ M

$M_e = 200$ kNm

$M_s = 503$ kNm

The ratio of mid span moment ignoring column flexibility to mid span moment with the inclusion of column flexibility, α' is:

486/503 = 0.966 i.e. error 3.2%

The value α' given by the beam line method (i.e. α' = 3.3%) is very close to the value given by equ. (5.26) which is 0.967, confirming the more general algebraic result.

Similarly to sub-frame SA1, in terms of stiffness equ. (5.27) becomes:

$$a/z = \frac{1}{\left[\dfrac{\alpha'}{(\alpha' - 1) + 1/\beta'}\right] - \beta'} \, . \, 3/8 \qquad (5.28)$$

where α' and β' are as defined previously.

Figures 5.11 and 5.12 show the effects of ignoring column flexibility on the value of beam mid span moment with the assumption of a linear M-Φ relationship for a header plate connection, for the general case of sub-frames similar to SB1 (i.e. fixed end columns). The limiting values x/a and z/a are also given in Table 5.3 in terms of flexibility and in Table 5.4 in terms of stiffness respectively.

5.3.3 Sub-frame analysis results

As a result of the analyses of the semi-rigid sub-frames, the following conclusions are reached:

i) The beam-line method is a very effetive tool for predicting the end restraint of a connection. However, due to length of calculation, the method is suitable only for simple sub-frames and becomes complex and difficult for more complicated sub-frames with different loading combinations.

ii) Table 5.5 summarises the analysis results for the sample calculation carried out. Values of beam end moments, M_e, and mid span moments, M_s, are tabulated for both conditions considered for the. far ends of the columns. For sub-frame SB1 which corresponds to columns far ends fixed, the values M_e and M_s are given between brackets. The assumption of a linear M-Φ relation underestimates the mid span moment by about 12% in comparison to the non-linear relationship. However, column flexibility has negligible effect on beam design moment for this particular case where x/a is 0.585 and z/a is 0.714.

iii) Certainly for connections stiffer than header plate connections, such as flush and extended end-plate connections, to base the linear M-Φ relationship on the initial stiffness would cause much larger errors in beam design moment. This is due to the fact that the latter connections (flush and extended end-plate) possess high initial tangent stiffness. This shows that one must use the full polynomial, or base the linear relation on a secant stiffness appropriate to the level of moment expected in the connection.

iv) Comparison of Figures 5.8 and 5.11 shows the effects of the column end conditions on beam design moment, M_s, assuming a linear M-Φ relationship for the connection. In both plots, limiting column flexibility rises as the connection to beam flexibility ratio increases. The comparison indicates that for a fixed error in mid span moment and for the same connection to beam flexibility ratio, the limiting column to beam flexibility ratio increases for sub-frames with fixed conditions at the far ends of the columns.

i.e. for example

say for a feasible error $\leqslant$ 10% and x/a = 0.4

z/a $\leqslant$ 1.26 for pin ends (from Table 5.1)

z/a $\leqslant$ 1.68 for fixed ends (from Table 5.3).

Therefore for this particular value z/a = 0.4, the column flexibility for a fixed ends sub-frame can be 26% more flexible than the pin ends sub-frame for an error say of 10% in beam design moment.

v) For that particular example, there is not much difference in beam end moments and mid-span moments between far end pinned and far end fixed as shown in Table 5.5. Values of M_e and M_s for sub-frame SA1 are exactly the same as for sub-frame SB1 when column flexibility is ignored.

vi) Similar boundary limits could be plotted for flush and extended end-plate connections.

5.4 Non-linear elastic analysis of semi-rigid frames:

The development of electronic computers has made it possible to write sophisticated structural analysis programs which include semi-rigid connections [5.17, 5.18, 5.19, 5.20, 5.21]. These methods are based on the systematic stiffness matrix method of structural analysis, rather than other methods. Earlier methods of analysis of semi-rigidly connected steel frames are described in Section 5.1 and a more comprehensive review of existing methods is given in an IABSE survey [5.22].

Various approaches and different degrees of non-linearity may be considered in the analysis. The present work is limited to a non-linear, i.e. second-order elastic analysis which takes into account the non-linear M-Φ relationship of the connection.

5.4.1 Linear elastic analysis

In a linear elastic analysis, a linear behaviour of material is assumed and no allowance is made for the influence of deformation of the frame due to axial forces (P- δ effects). Therefore the effect of deformation is disregarded in the equilibrium equations. The analysis leads then to the solution of a set of linear equations in the usual forms:

$$\underline{E} = \underline{K} \cdot \underline{X} \tag{5.29}$$

where $\underline{E}$ is the externally applied vector, $\underline{K}$ is the overall stiffness matrix of the structure derived from the slope deflection equation and $\underline{X}$ is the unknown joint displacements of the structure.

The elastic stiffness matrix $\underline{K}$ takes into account the connections rigidity and possibly the joint size. Conventional computer programs for the analysis of frames with rigid joints can be easily modified to allow for joint flexibility by means of an appropriate correction matrix to be applied to the element stiffness matrix and the fixed end-force vector.

If a linear M-Φ relationship is assumed, the method of analysis requires neither an incremental nor an iterative approach. In this case, the initial tangent stiffness of the connection is implemented in the analysis as described in the earlier pre-computer methods [5.1, 5.2, 5.23, 5.24, 5.25, 5.26] using slope-deflection equations or the moment-distribution method or based on the matrix displacement procedure as described in the earlier investigations [5.4, 5.5, 5,6, 5.8, 5.9, 5.10, 5.27].

As previously discussed, the linear M-Φ relationship is strictly applicable for very low values of moments. It becomes increasingly inaccurate as the moment increases. Therefore, the use of the initial tangent stiffness leads to an overestimation of the stability of the frame and an underestimate of lateral deflection in unbraced frames. A more accurate analysis will result from inclusion of non-linear joint behaviour. Such analysis can be performed either incrementally or iteratively.

5.4.1.1 *Incremental analysis*

If the only source of non-linearity is the M-Φ relationship, the analysis can be carried out in an incremental fashion as a sequence of linear analyses. At each step, the stiffness matrix is modified to account for changes in the joint rigidity by using the tangent stiffness. This method uses the last obtained values of moments to find an appropriate tangent stiffness, and then iterates on the tangent stiffness until acceptable moment tolerance is met on the current load step. A linear elastic (i.e. first-order) analysis using an incremental procedure with a bilinear M-Φ relationship is discussed in reference [5.10].

5.4.1.2 *Iterative analysis*

Alternatively, an iterative procedure can be used, with the joint behaviour represented in secant stiffness form. The secant stiffness is a linear relation between the moment and the rotation at the bending moment considered. The secant stiffness is the only parameter assumed necessary to allow for the non-linearity of the joint behaviour. This assumption leads to a basically linear elastic analysis, though iteration about this parameter is required. The secant stiffness iteration process is shown in Figure 5.13. A procedure of this type is discussed in references [5.13, 5.14].

5.4.2 Non-linear elastic analysis

In first-order elastic analysis it is assumed that the deformations are small and the stiffness of a member remains unchanged during the complete history of loading. However, in a second-order elastic analysis, the effect of member axial force on its stiffness is included. Incremental or iterative approaches can be modified to allow for the influence of deformation of the frame, thereby giving second-order analysis procedures, such as those reported in references [5.11, 5.12, 5.13, 5.14, 5.19]. If an iterative procedure is used to include the joint flexibility in the analysis, a second iteration is required to allow for geometric effects.

In the above discussed methods of analysis only the aspects which are related to the joint flexibility are discussed.

A comparison of these two methods reveals that the tangent stiffness approach controls the error at a local incremental level [5.27, 5.28], and normally requires the use of a small increment in order to obtain sufficiently accurate results. On the other hand, the secant stiffness approach is not sensitive to the loading history of the frame and a large enough increment can be used to limit the required number of iteration cycles for convergence. Another difference between these two methods is that in the tangent stiffness method an extra matrix called the geometric stiffness matrix, which is sensitive to the local slope of the M-Φ curve of a member, is used [5.29]. As a result of the previously discussed investigations, the tangent stiffness is usually used with an incremental load application technique while the secant stiffness should be used in an analysis where the full factored load is applied in one step. Therefore, the secant stiffness approach will be used in the present study for the following reasons:

a) if an initial stiffness is used the deflections derived will be erroneous;

b) the tangent stiffness is too laborious and expensive to use [5.30];

c) the tangent stiffness is time consuming because the method itself is prone to accumulation of sizeable error from each load step, unless the step size is kept very small;

d) the factored loads will be applied in one step [5.31];

e) the secand stiffness provides an integrated average of how the connection arrived at the present level of loading [5.31];

f) the computer program developed by the present author to analyse non-linear elastic semi-rigid frames uses a secant stiffness approach because the original programs [5.32, 5.33] used stability functions for second-order effects and these require analysis under total load.

5.5 The proposed method of analysis

The analysis program developed by the present author is an extension of an existing computer program for non-linear elastic analysis of very large rigidly connected plane frames described in references [5.32, 5.33, 5.34]. This is based on the matrix displacement method of analysis in which the unknown joint displacements are obtained by solving the matrix equation (equ. (5.29)). The second-order effects are taken into account by using the stability functions introduced by Livesley [5.35]. These functions depend on the ratio of axial force to Euler load of a member. Majid and Anderson [5.34] make use of a compact elimination technique due to Jennings [5.36]. This utilises the symmetrical feature of the stiffness matrix and therefore stores only the elements which lie between the first non-zero one and that on the leading diagonal, inclusive, for each row of the stiffness matrix. All elements preceding the first non-zero element in each row are not stored. A Gauss-elimination method is used to solve the set of linear simultaneous equations.

Anderson and Lok [5.19] developed an analysis program basically similar to the one described by Majid and Anderson [5.34]. In [5.19], the deformation of the joints is allowed by iteratively revising the load vector defined by equ. (5.29), the overall frame stiffness matrix remaining unchanged. Savings in the required computer time and storage are then made possible. The non-linear M-Φ relationship of the semi-rigid connections are represented by Frye and Morris [5.13] polynomial equations which are implemented in the analysis program. The program was converted by this author from Algol to Fortran 77, using M-Φ equations [5.13] expressed in metric units. While using this program, great difficulties with convergence were experienced due to the nature of the method used in the analysis program. The method of analysis was found to be suitable only for frames with very stiff semi-rigid connections, where convergence was achieved in a few number of iterations.

The program was therefore rewritten using successive estimates of the secant stiffness of the connections as shown in Figure 5.13, instead of fixing the rotation of the connection before each iteration as proposed by Anderson and Lok [5.19]. The reasons for using the secant stiffness approach are given in the previous section of this chapter. The program permits multi-linear representation of connection behaviour. The secondary effects of axial load on column stiffness are included by calculating stability functions from the previous iteration. The solution procedure therefore iterates about both the secant stiffness and the axial forces. The solution is assumed to have converged when the difference in the secant stiffness first and then axial forces between two successive iterations is less than a prescribed tolerance limit, fixed by the author as 0.1%.

The advantage of this analysis program is the number of iterations required for convergence is reduced by more than 50% compared to the Anderson and Lok [5.19] program. The second advantage is the analysis program always converges unless the frame has already completely lost stiffness at load level less than that being considered.

The small-deformation theory is assumed in the analysis.

5.6 Overall stiffness matrix

In the present author's procedure, the semi-rigid connections are considered as elastic hinges.

Consider now a member i-j of a structure with semi- rigid connections at both ends as shown in Figure 5.14. Joints i and j are respectively the first and second end of the member i-j. This is indicated by the direction of the arrow on the member. The clockwise end moments acting on the member are considered positive. The effect of such actions will cause a deformed shape with reverse curvature as shown in the figure.

As a result of the semi-rigid connection at end i, the rotation of the member at i becomes the sum of the joint rotation θ_i without a hinge and the additional rotation Φ_i due to deformation of the connection. Therefore a semi-rigid connection gives the frame an extra degree of freedom Φ. As the external load vector is equivalent to the joint displacement vector there is therefore a corresponding element in the load vector, given by M_h. As the moment M_h also equals the moment at the end of the member, the member i-j contributes the terms shown in Figure 5.15 to the overall stiffness equations $\underset{\sim}{E} = \underset{\sim}{K} \cdot \underset{\sim}{X}$.

The terms in the stiffness matrix are defined as follows:

$a = EA/L$

$b = 12\ EI\varphi_5/L^3$

$d = -6\ EI\varphi_2/L^2$

$e = 4\ EI\varphi_3/L$

$f = 2\ EI\varphi_4/L$

s,c, correspond respectively to the direction sine and cosine of the angle of inclination of the member measured clockwise positive from the first end i. E, I, A and L are Young's modulus of elasticity, second moment of area, cross sectional area and length of the member i-j. φ's are the stability functions defined in references [5.32, 5.35] to take into acount second-order effects.

As a result of the deformation of the connections, the member shown in Figure 5.14 will tend to 'relax' and straighten out. Thus the rotations of the connections will be anticlockwise. It follows that the relationship between the bending moment M_h at the semi-rigid connection and the rotation is:

$$M_h = -k\Phi \tag{5.30}$$

where k is the secant stiffness as shown in Figure 5.13. Thus, it can be assumed that a semi-rigid hinge behaves like a plastic hinge with its plastic hinge moment M_h being defined by equ. (5.30).

Thus for the semi-rigid connections at i and j:

$$M_{hi} = -k_i\Phi_i \tag{5.31}$$

$$M_{hj} = -k_j\Phi_j$$

When these are substituted into the load vector shown in Figure 5.15, the stiffness equations can be re-arranged to the form shown in Figure 5.16. Thus for an assumed value of the secant stiffness each semi-rigid connection contributes one additional unknown, Φ, and one additional equation to the overall load-deflection equations for the structure (equ. (5.29)).

It should be noticed that in Figure 5.15 the coefficients of θ_i, Φ_i of end i and θ_j, Φ_j for end j are exactly the same. Thus, inserting semi-rigid connection at the ends of a member cause the elements of the member contributions, corresponding to θ_i and θ_j in the overall stiffness matrix, to be repeated as elements corresponding to the hinge rotation Φ_i and Φ_j. Each sub-matrix K_{ii}, K_{ij}, K_{ji} and K_{jj} contains a 4 x 4 element matrix. When the rows and columns corresponding to those connections are deleted from Figure 5.15 the stiffness equations will be identical to a rigidly jointed member. The contribution of other members connected to joint i and j are similarly obtained. The size of the general stiffness matrix is of the order (3m+n), where m is the total number of rigid joints in the frame and n is the number of real hinges and semi-rigid connections. Accordingly the load vector F consists of (3m+n) elements of which the 3m elements are the external loads and the n elements are the hinge moments M_h for a total of n hinges in the frame. With reference to Figure 5.17 showing a four storey single bay frame, each numbered joint is considered in ascending order for the construction of the overall stiffness matrix. The total stiffness of the joint is the sum of the individual member stiffnesses connected to that joint. Therefore, non-zero sub-matrices, K_{ii} etc., will populate the overall stiffness matrix only at locations corresponding to joint interconnections. Hence, with reference to Figure 5.17, the overall stiffness matrix is seen to contain many zero sub-matrices, and the non-zero sub-matrices are directly related to the joint connection list. Further, K is symmetric along the leading diagonal.

The method of Jennings [5.36] makes use of the symmetry of the overall stiffness matrix for the storage and rapid solution of the stiffness equations. Only the irregular half band-width, outlined in Figure 5.17, is stored and operated on by the compact elimination technique.

5.7 Program procedure

The data for the program and the manner in which the overall stiffness matrix is constructed are similar to those for the non-linear elastic analysis program with rigid joints described by Majid and Anderson [5.34].

The program procedure is as follows:

1. Read input data, including piece-wise M-Φ behaviour for the semi-rigid connections.
2. Construct the overall stiffness matrix, using Figure 5.16 as the basis.
3. Solve the overall stiffness equations for the unknown displacements and calculate member end forces and moments.

If every semi-rigid joint is represented by a linear M-Φ relationship, then the solution does not require any iteration, unless a second-order analysis is required. For piece-wise linear behaviour, the procedure continues.

4. Re-calculate k for each semi-rigid connection, using $k = M_h/\Phi$. The recalculated value is denoted k_1.
5. Apply tolerance test on successive values of k_1 for each semi-rigid connection.
6. (i) If convergence has been achieved, apply a second tolerance test on successive values of axial force in each member. If this test is not satisfied then the stability functions are re-calculated and the procedure repeated from step 2. If the test on axial force is satisfied then the procedure is terminated and the results are output.

   (ii) If the tolerance test on secant stiffness is not satisfied then usually the new value k_1 is used and the procedure repeated from step 2. However, the new value k_1 is also compared with that obtained two iterations before (i.e k_3). If k_1 and k_3 for a particular joint satisfy the tolerance test then the stiffnesses are oscillating. In this case the new secant stiffness is taken not as k_1 (i.e. present secant stiffness) but as the average for the two previous iterations. The procedure is then repeated from step 2 with the revised values of secant stiffness.

Throughout the analysis, the stability of the frame is checked by calculating the determinant of the overall stiffness matrix. If first-order analysis is required the stability functions are set to unity to avoid reductions in member stiffness due to compressive axial force.

The success of the procedure in terms of speed of analysis and size of frame that can be dealt with owes much to the use of a compact form of the stiffness matrix due to Jennings [5.36].

The process and the program procedure are described with the aid of the flow diagram in Figure 5.18.

5.8 Iteration process and convergence problems

Many difficulties were experienced in convergence while developing the present computer analysis program using a non-linear M-Φ relationship of connection behaviour. The first problem faced was when using the procedure by Anderson and Lok [5.19]. As discussed previously in section 5.6, the analysis converged only for frames with very stiff connections. As a result of this, an iterative procedure based on the secant stiffness was used, instead of fixing the rotation of a connection before each iteration as described in reference [5.19]. The reasons for choosing the iterative analysis procedure using the secant stiffness are given in section 5.4.

The non-linear M-Φ curve is represented in the form of a series of straight lines. The initial estimate of the secant stiffness for starting the computational process can be set to any value read from the given data series (M_m, Φ_m). Ideally, this stiffness should correspond to the bending moment in the connection, but when the computational process starts, the moment is unknown. A good initial estimation of the secant stiffness can reduce the number of cycles of iteration and could be made by using the beam-line concept. In practice though the present author has taken the initial estimate to be the slope of the first part of the piece-wise linear M-Φ relation (i.e. $k_1 = M_1/\Phi_1$). This stiffness was found to converge with less number of iterations in comparison to using the last secant stiffness (i.e. $k_1 = M_m/\Phi_m$) taken from the M-Φ data.

To describe the iteration process, consider a structure whose connections have non-linear moment-rotation characteristics. The moment-rotation function for a typical connection is illustrated in Figure 5.13, and has the form

$$\Phi = f(M) \tag{5.32}$$

where f(M) is a non-linear function of the moment acting on the connection idealised as piecewise linear behaviour. The analysis procedure is begun by replacing the non-linear moment-rotation function for the connection considered, by a linear relationship of the form:

$$M = k_1 \Phi \tag{5.33}$$

As illustrated in Figure 5.13, k_1 represents the initial slope of the first part of the piecewise-linear M-Φ curve which can be regarded as the secant stiffness for values M and Φ very close to the origin. The moment-rotation relationships for all other connections considered in the structure are similarly linearised.

After constructing the overall stiffness matrix $\underline{K}$, based on the initial estimates of connection stiffness, the stiffness equations are solved:

$$\underline{X} = \underline{K}^{-1} \cdot \underline{L} \tag{5.34}$$

Thus, the resulting displacements are determined and hence member end moments can be calculated. The calculated moment using k_1 is M_1 and the connection rotation given by the analysis is Φ_1. However, the rotation calculated from the M-Φ relationship (i.e.Φ'_1 = $f(M_1)$) is Φ'_1. A better approximation to the connection moment-rotation function is thus seen to be:

$$M = k_2\Phi \tag{5.35}$$

where $k_2 = M_1/\Phi'_1$ as illustrated in Figure 5.13. Equation (5.34) and similar relationships for all connections are then used to calculate the new member force-displacement relationships and a second iteration of the analysis is performed. A new moment, M_2, is found to occur at the typical connection and the corresponding connection rotation, as shown in Figure 5.13, is

$$M_2 = K_2\Phi_2 \tag{5.36}$$

Again the connection rotation calculated from equ. (5.32) is:

$$\Phi'_2 = f(M_2) \tag{5.37}$$

Hence, a third linear relationship, which will lead to better approximation to the correct moment end rotation at the connection is

$$M = k_3\Phi \tag{5.38}$$

where $k_3 = M_2/\Phi'_2$. For simplicity, this is not shown in Figure 5.13.

The above procedure is repeated until convegence has been achieved. This is done by applying a tolerance test on two consecutive stiffnesses.
A counter is set in the routine procedure and the iterative process is terminated when the total accumulated number of iterations exceeds the prescribed upper limit which is set to 100.

In most cases it was found that k_i and k_{i+1} are within the tolerance test after few iterations for frames with stiff or relatively stiff connections. However for frames with flexible or moderately flexible connections, convergence is achieved at a higher number of iterations in comparison to stiffer structures.
Convergence problem arises when very flexible connections are used or when the

frame collapses at a load level close to that prescribed for the analysis. For such frames, either the analysis did not converge after 100 iterations or the bending moment was out of the range of the M-Φ relation. In the latter case the bending moment given by the analysis for at least one of the semi-rigid connections exceeds the maximum moment capacity of the connection defined by the M-Φ data. To overcome this problem, a substitution of this out of range value of moment is done by replacing it by the maximum value of moment obtained from the given M-Φ data. This is shown in Figure 5.19. This revised stiffness was found to solve the problem in some cases.

However for the other most common convergence problem, i.e. where the analysis did not converge after 100 iterations, the secant stiffnesses oscillated from a low value in one iteration to a very high value in the following iteration or vice versa. This behaviour could lead to an extreme value of connection stiffness which sometimes could give rise to a value of moment higher than the maximum offered by the connection. In order to achieve convergence or to accelerate the computation, a tolerance test on secant stiffness was devised to check whether the new value k_i and the value k_{i-2} (i.e. secant stiffness obtained two iterations before the present one) are similar. If k_i and k_{i-2} for a particular joint satisfy the tolerance test then the stiffnesses are taken to be oscillating. In this case the new secant stiffness is taken not as k_i (i.e. present secant stiffness) but as the average for the two previous iterations (i.e. $(k_{i-1}+k_{i-2})/2$). By adopting such a procedure, the problem of exceeding the maximum moment provided by the M-Φ data as well as the convergence problem was solved.

5.9 Numerical example

The simple fixed-end beam with its numbering system is shown in Figure 5.20. This simplified structure represents a first floor beam of a three storey one bay frame. The beam is connected to the columns by an unstiffened extended end-plate connections at both ends. In the interests of simplicity the columns are omitted and the beam is assumed to have fixed supports. The beam ends are connected to the supports by the connections.

The moment-rotation relationship of the connection is determined by the modified Frye and Morris prediction equation (equ. (4.11)) and is given by:

$$\Phi = 1.048 \times 10^{-6}M - 1.956 \times 10^{-14}M^{3} + 3.937 \times 10^{-22} \times M^{5} \qquad (5.39)$$

where M is in kNcm and Φ is in radians.

The analysis results are tabulated in Figure 5.20 for semi-rigid and rigid assumptions. For semi-rigid analysis, two calculations are presented. The first took the initial estimate of secant stiffness as the slope of the initial part of the M-Φ data. The second took the initial estimate as the stiffness corresponding to attainment of the maximum resistance moment of the connection. For each iteration the beam end moment M_e, mid span moment M_s, mid span deflection y and the connection rotation are tabulated in Figure 5.20. For that simple example, the analysis converges after five iterations when using the last secant stiffness and four iterations if the initial secant stiffness is used as the initial estimate of the secant stiffness.

The vertical displacement at joint number 2 is increased by 119% in comparison with fully rigid analysis. The final bending moment distribution indicates a 39.8% reduction in beam end moment, while the sagging moment at mid span is increased by the same amount.

Using the beam-line method shown in Figure 5.21 for this particular simple example, the graphical method converges to:

$M_e \quad = \quad 48.0$ kNm

$\Phi_e \quad = \quad 3.885 \times 10^{-3}$ radians

The values read from Figure 5.21 are in good agreement with the computed values.

The M-Φ curve shown in Figure 5.21 predicted by the modified Frye and Morris equation (equ. (4.11)) presents a reverse curvature. To overcome this problem, the initial part of the curve where reverse curvature appears can be approximated by straight line as previously discussed in Chapter 4, section 4.5.2.1. This is shown in Figure 5.21 by the dashed line.

5.10 Analysis of the calibrating frames

The accuracy and the validity of the proposed method is checked by analysing three frames specified by Zandonini [5.37] for an ECCS document devoted to the stability of semi-rigid frames. Several universities were asked to contribute to this document by using their available computer programs to analyse the three frames shown in Figure 5.22 in order to check the consistency in the prediction of frame response obtained from the different numerical approaches. The different research groups are:

i) Zandonini (Milan)

ii) Stutzki (Aachen)

iii) Tschemmernegg (Innsbruck)

iv) Colson (Cachan)

v) Ohta (Warwick)

5.10.1 Frames data

The frame geometry and loading conditions are presented in Figures 5.22a, 5.22b and 5.22c for Frames A, B an C respectively.

Frames A and B are unbraced and differ only in geometry. However, Frame C is braced and has also a different loading condition, with vertical forces applied to the columns at the roof level.

In addition to the stated horizontal load, horizontal forces due to geometrical imperfections as specified by Eurocode 3 [5.38] are added. Geometrical imperfections of the three frames are shown in Figure 5.23.

All members are in steel of grade Fe 360 with a yield strength of 235 N/mm² and a Young's modulus of elasticity of 210 kN/m². The stress-strain relationship is idealised as elastic-perfectly plastic. Strain hardening is not taken into account.

The connections used were extended end-plates with backing plates and are shown in Figure 5.24. The moment-rotation curves of the connections were determined by the macro-mechanical model described in reference [5.39]. Their moment-rotation curves were represented by the piece-wise linear relations shown in Figure 5.25.

The specified uniformly distributed loads of the frames were approximated by two types of point loads. In type 1 loading, the uniformly distributed load is replaced by point loads over the span of the beam member; in type 2 loading multiple point loads are used, as shown in Figure 5.26, 5.30 and 5.34 for Frame A, B and C respectively.

The behaviour of Frames A, B and C with the specified loads are shown in Figures 5.26 to 5.40 for both first and second order analysis. The effects of semi-rigid joint action on frame behaviour will be discussed in the next chapter, including comparison with analyses assuming rigid joints. The first order analysis results will be used in Chapter 6 to study criteria for design of semi-rigid steel frames.

5.10.2 Comparison of different pattern of loading

Comparing the results for type 1 and type 2 loading on the semi-rigid analysis (Figures 5.28, 5.32 and 5.36 for bending moment distribution and Figures 5.29, 5.33 and 5.37 for deflected shape), type 1 loading underestimates the larger beam end moments by 7.6%, 7.7% and 17.8% and overestimates the sagging moment by 9.4%, 3.9% and 8.6% for Frames A, B and C respectively. The axial forces are the same. However the sway at the top of the structure is underestimated with type 1 loading by 4.7% and 17% for the unbraced Frames A and B respectively. For the braced frame, Frame C, type 1 loading underestimates the beam deflection by 8.6%.

As a result of this comparison, when strength is considered the uniformly distributed loading can be approximated by type 1 loading. In this case, the design beam moment is overestimated by less than 10% in comparison to the original specified loading. However, if serviceability limit is considered, the sway of the frame should be determined by considering type 2 loading.

Type 2 loading required more computational time because more joints are included in the frame. This also means more members. In the case of Frame A, the size of the overall stiffness matrix is 51 and the locations required by the compact stiffness matrix is 534 terms, instead of 318 locations required by type 1 loading. Furthermore, type 1 loading needs only 7 iterations to converge, whereas 46 iterations were required in the case of type 2 loading; in both cases the analyses took into account second-order effects for the same tolerance of 0.1% on connection secant stiffness and axial force.

5.10.3 Comparison of analysis results with other researchers

The accuracy of the proposed method developed by the present author is assessed by comparing the analysis results with the various analyses developed by the earlier mentioned researchers.

The lateral deflection is shown in Figures 5.38 to 5.40 for Frames A, B and C respectively. As the developed computer analysis program is limited to second-order elastic analysis, results are shown only up to load level one prescribed for the analysis, to avoid inelastic behaviour in the frames. The bending moments values and the lateral sway were found very close to other set of results, except those of Tschemmemegg amd Ohta. As good agreement was obtained with results from the other research workers, it is concluded that the accuracy of the present author's program is satisfactory.

5.11 Conclusions

The beam-line method is a very effective tool for predicting the end restraint of a connection. However, due to length of calculation, the method is suitable only for simple sub-frames.

Based on studies of single bay sub-frames where the far ends of the columns are either fixed or pinned and when header plate connections are used, the following conclusions were reached:

i) The assumption of a linear M-Φ relationship underestimated the beam design moment by a considerable amount and cause much larger errors for a beam with a high degree of end restraint.

ii) Limits of column to beam flexibility and connection to beam flexibility have been determined for a given error in beam design moment. Within these limits column flexibility may be ignored.

A well established computer program for second-order frame analysis has been extended to frames with semi-rigid connections. Successive estimates are made of the secant stiffness of these connections, to represent their effect on frame behaviour. The non-linear M-Φ curve of the semi-rigid connection is idealised as a piece-wise linear curve. Any type of connection, pinned, rigid or semi-rigid can be analysed by the program.

The validity of the program is checked by analysing three frames specified by an ECCS task group in order to examine consistency in the prediction of frame response from five different research groups. The author's results are in good agreement with those of other researchers.

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α'	x/a [conn:beam]$_{flex}$	β'=3x/a+1.5	z/a [column:beam]$_{flex.}$
0.90	0.100	1.800	0.351
	0.150	1.950	0.460
	0.200	2.100	0.585
	0.250	2.250	0.726
	0.300	2.400	0.884
	0.350	2.550	1.061
	0.400	2.700	1.258
	0.450	2.850	1.475
	0.500	3.000	1.714
0.95	0.100	1.800	0.158
	0.150	1.950	0.205
	0.200	2.100	0.258
	0.250	2.250	0.317
	0.300	2.400	0.382
	0.350	2.550	0.453
	0.400	2.700	0.531
	0.450	2.850	0.615
	0.500	3.000	0.706

Table 5.1 Analysis results of sub-frame SA1 in terms of flexibilities

α'	a/x [conn:beam]$_{stif.}$	β'=3x/a+1.5	a/z [column:beam]$_{stif.}$
0.90	10.000	1.800	2.849
	6.667	1.950	2.174
	5.000	2.100	1.709
	4.000	2.250	1.377
	3.333	2.400	1.131
	2.857	2.550	0.942
	2.500	2.700	0.795
	2.222	2.850	0.678
	2.000	3.000	0.583
0.95	10.000	1.800	6.329
	6.667	1.950	4.878
	5.000	2.100	3.876
	4.000	2.250	3.155
	3.333	2.400	2.618
	2.857	2.550	2.207
	2.500	2.700	1.883
	2.222	2.850	1.626
	2.000	3.000	1.416

Table 5.2 Analysis results of sub-frame SA1 in terms of stiffnesses

α'	x/a [conn:beam]$_{flex.}$	β'=3x/a+1.5	z/a [column:beam]$_{flex.}$
0.90	0.100	1.800	0.469
	0.150	1.950	0.614
	0.200	2.100	0.780
	0.250	2.250	0.968
	0.300	2.400	1.179
	0.350	2.550	1.415
	0.400	2.700	1.677
	0.450	2.850	1.966
	0.500	3.000	2.286
0.95	0.100	1.800	0.211
	0.150	1.950	0.274
	0.200	2.100	0.344
	0.250	2.250	0.423
	0.300	2.400	0.509
	0.350	2.550	0.604
	0.400	2.700	0.707
	0.450	2.850	0.820
	0.500	3.000	0.941

Table 5.3 Analysis results of sub-frame SB1 in terms of flexibilities

α'	a/x [conn:beam]$_{stif.}$	β'=3x/a+1.5	a/z [column:beam]$_{stif.}$
0.90	10.000	1.80	2.135
	6.667	1.95	1.629
	5.000	2.10	1.282
	4.000	2.25	1.033
	3.333	2.40	0.848
	2.857	2.55	0.707
	2.500	2.70	0.596
	2.222	2.85	0.509
	2.000	3.00	0.437
0.95	10.000	1.80	4.739
	6.667	1.95	3.650
	5.000	2.10	2.907
	4.000	2.25	2.364
	3.333	2.40	1.965
	2.857	2.55	1.656
	2.500	2.70	1.414
	2.222	2.85	1.219
	2.000	3.00	1.063

Table 5.4 Analysis results of sub-frame SB1 in terms of stiffnesses

Column flexibility / M-Φ	With column flexibility	Ignoring column flexibility
M-Φ linear	M_e = 194.5 (200) M_s = 508.6 (503)	M_e = 217 (217) M_s = 486 (486)
M-Φ non-linear	M_e = 144 (146) M_s = 559 (557)	M_e = 150 (150) M_s = 553 (553)

Analysis results for sub-frame SB1 are shown between brackets.

Table 5.5 Values of beam end moments and mid span moments for sub-frames SA1 and SB1.

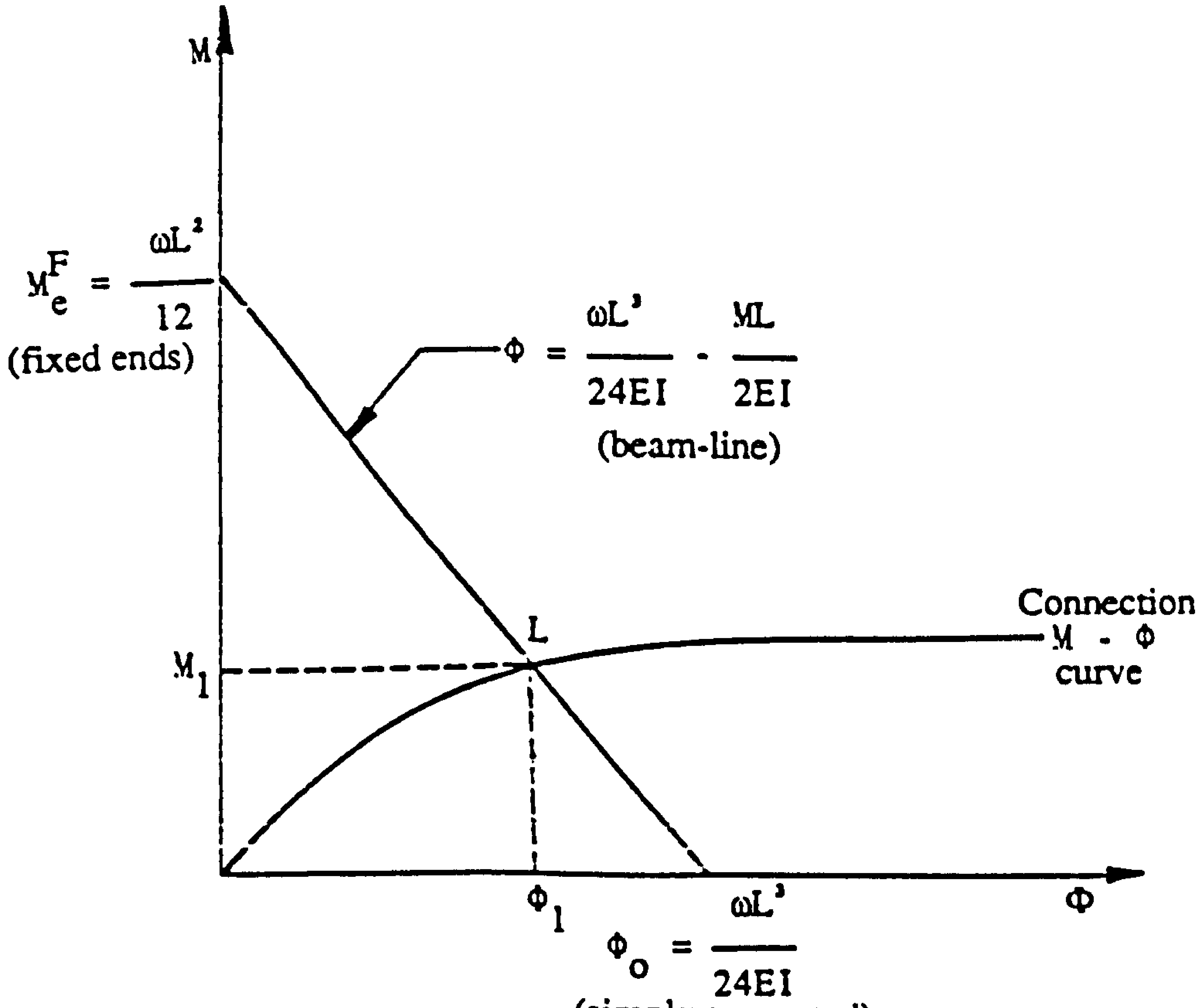


FIG.5-1 Conventional beam line.

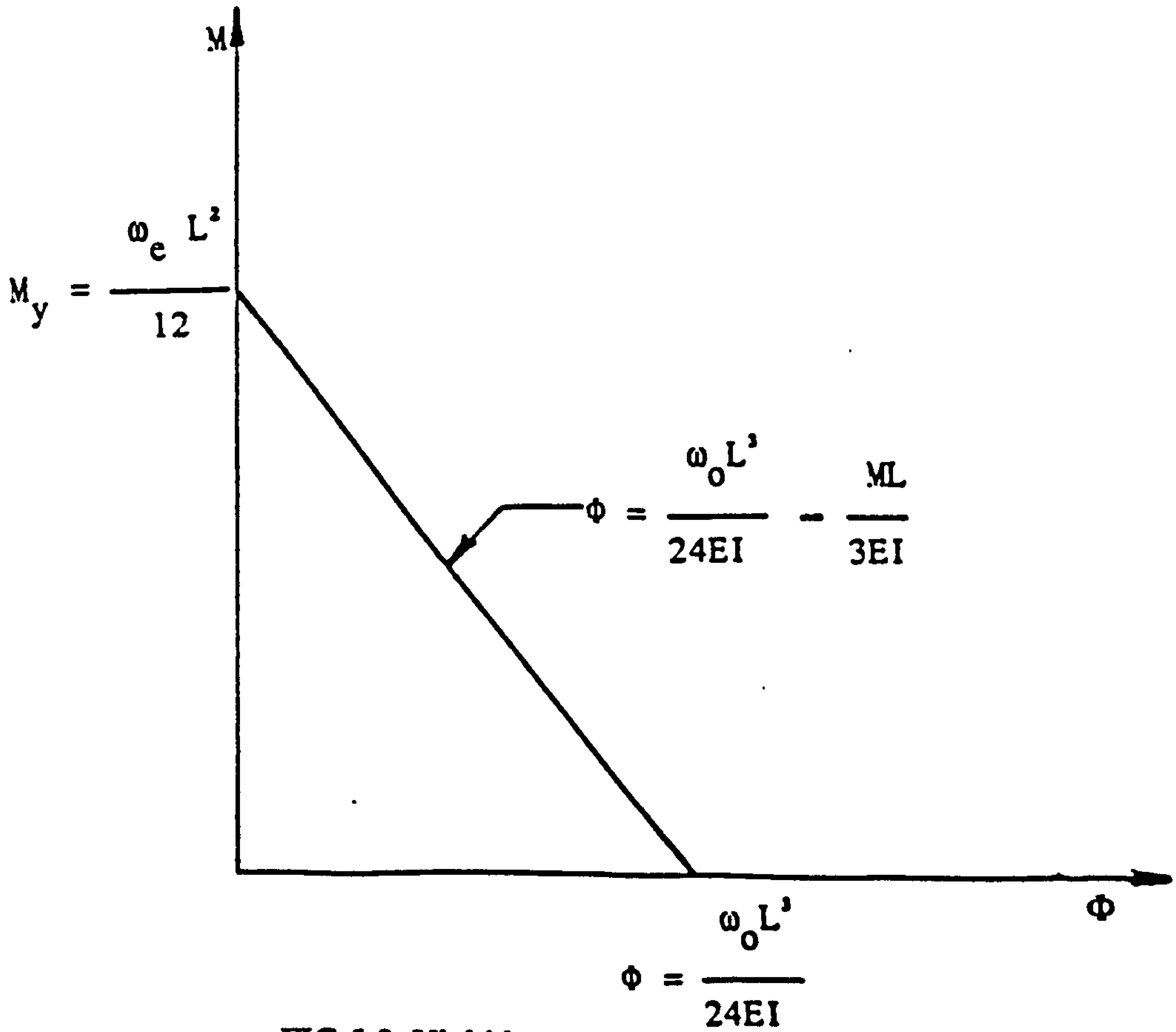


FIG.5-2 Yield beam line.

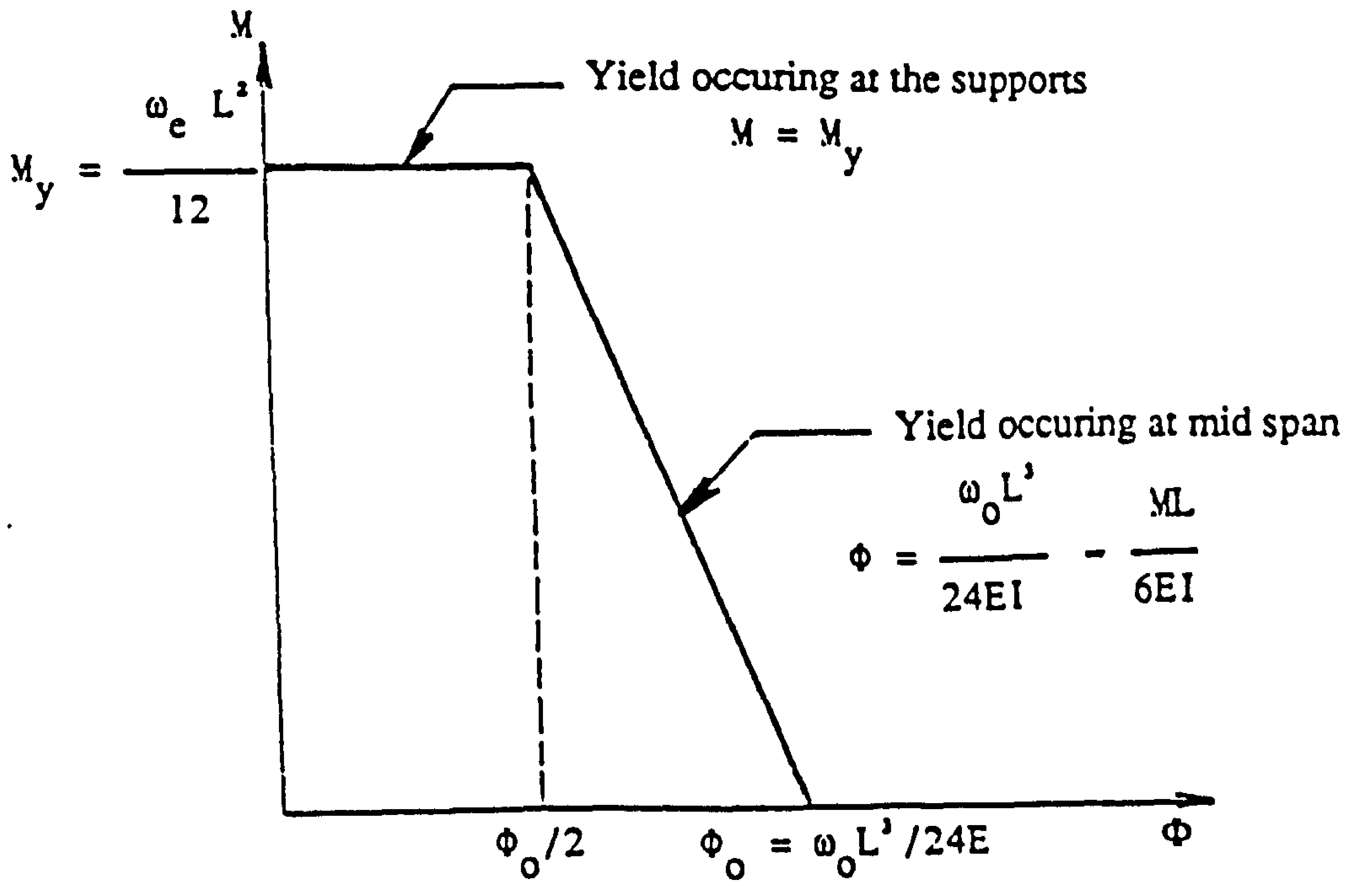


FIG.5-3 Two-phase beam line.

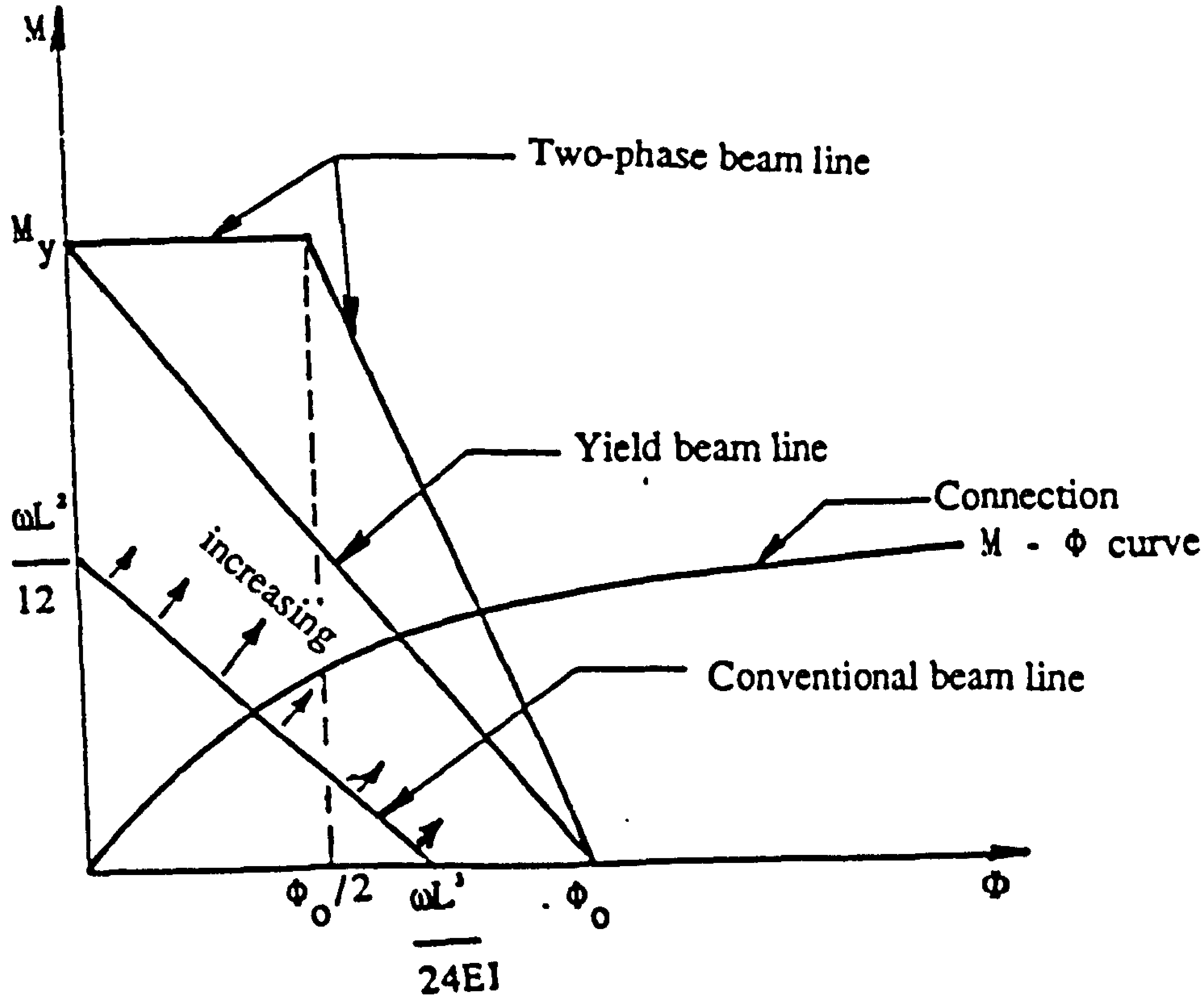
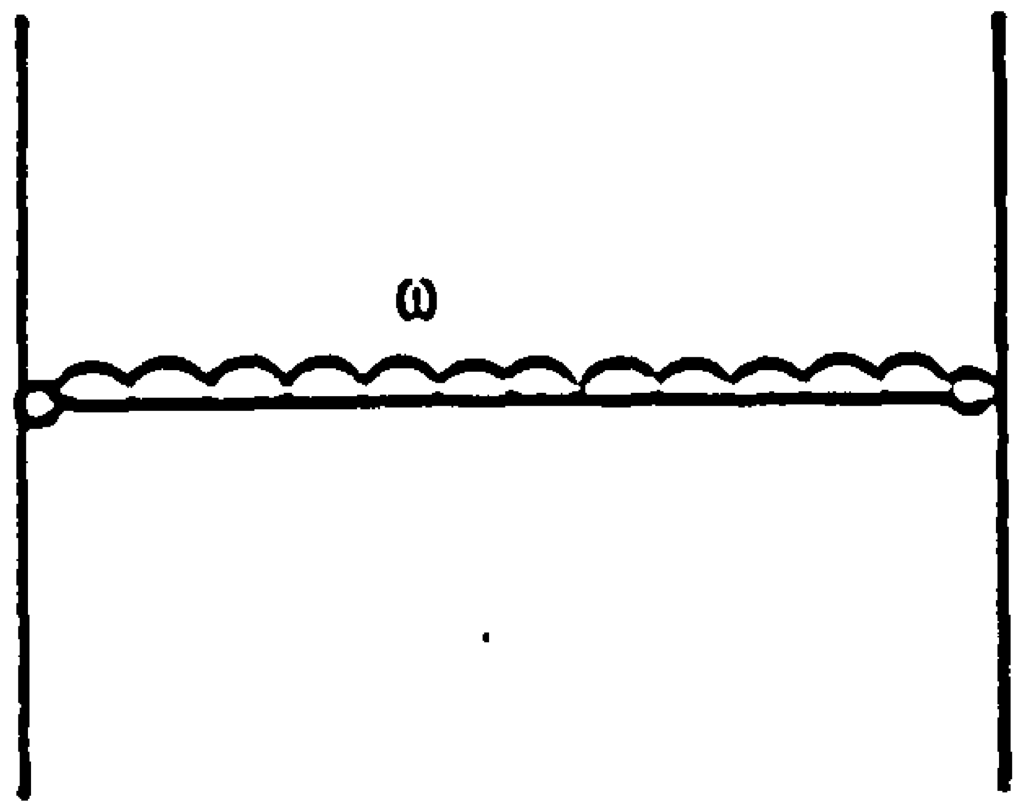
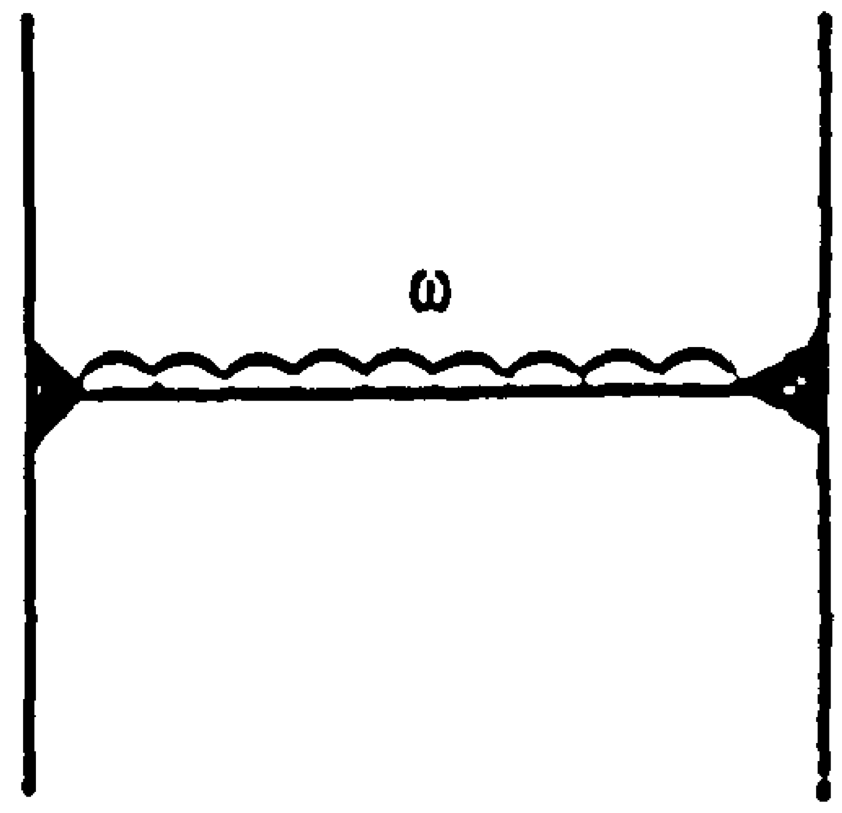


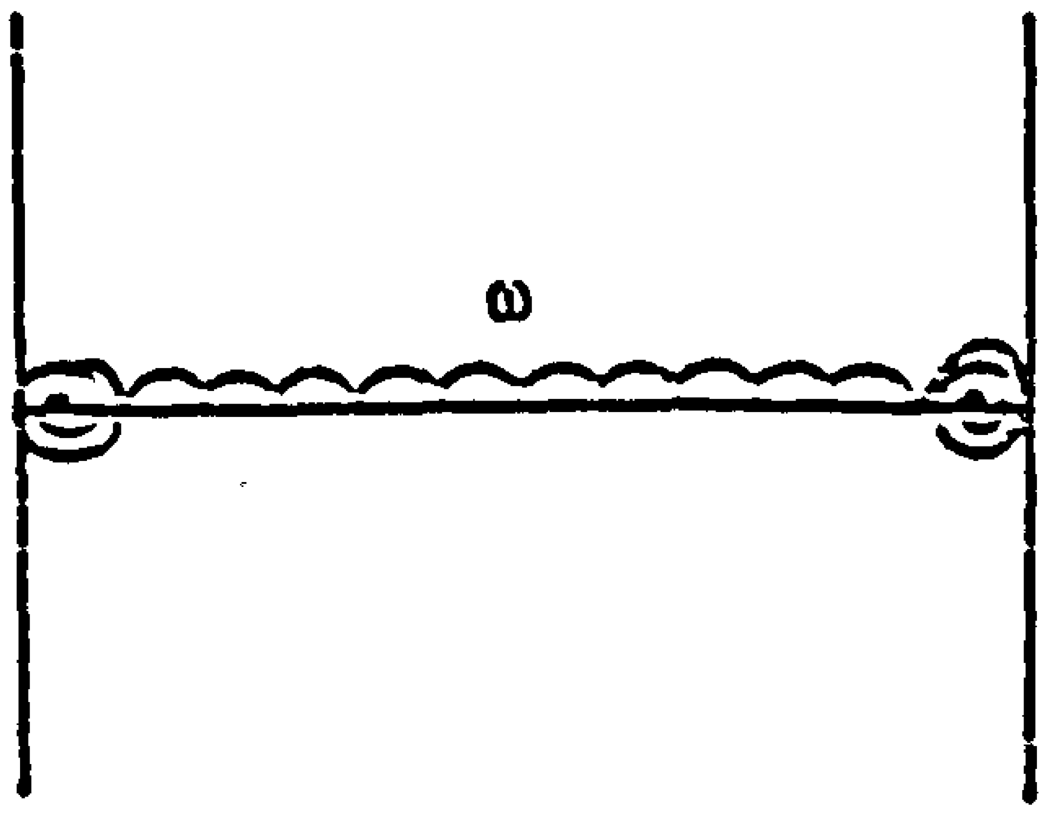
FIG.5-4 Comparison of various beam lines.



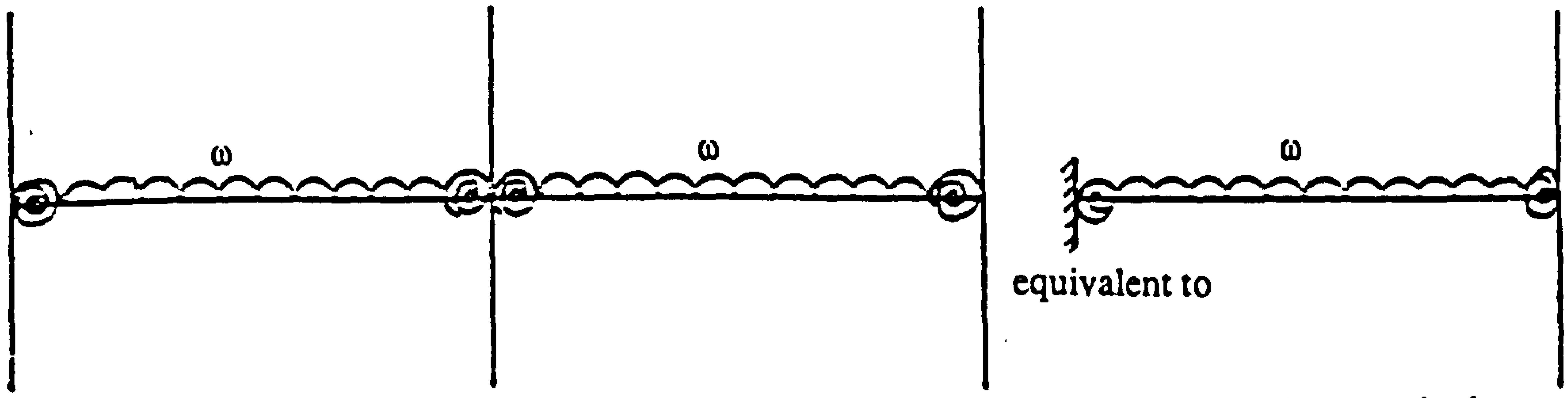
a) Simple conection in sub-frame.
Order of design:
i) beams
ii) columns



b) Rigid connections in sub-frame.
Order of design:
i) guess column sizes
ii) beams
iii) possible need to iterate

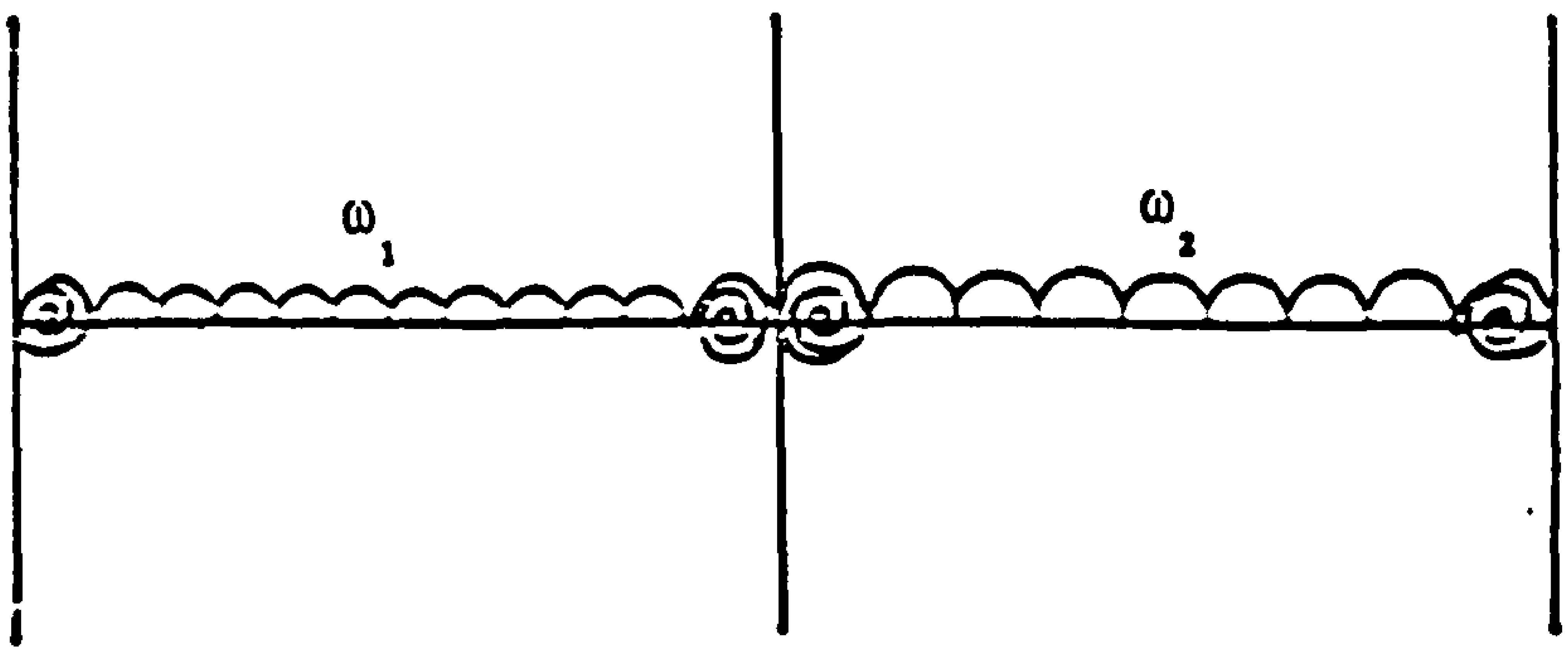


c) Semi-rigid connections in single bay frame.



d) Semi-rigid connections in multi-bay frame.
with equal spans and equal loading.

e) Equivalent single
bay sub-farme



f) Semi-rigid connections in multi-bay frame
with equal spans and inequal loadding.

FIG.5.5 Sub-frames analysis.

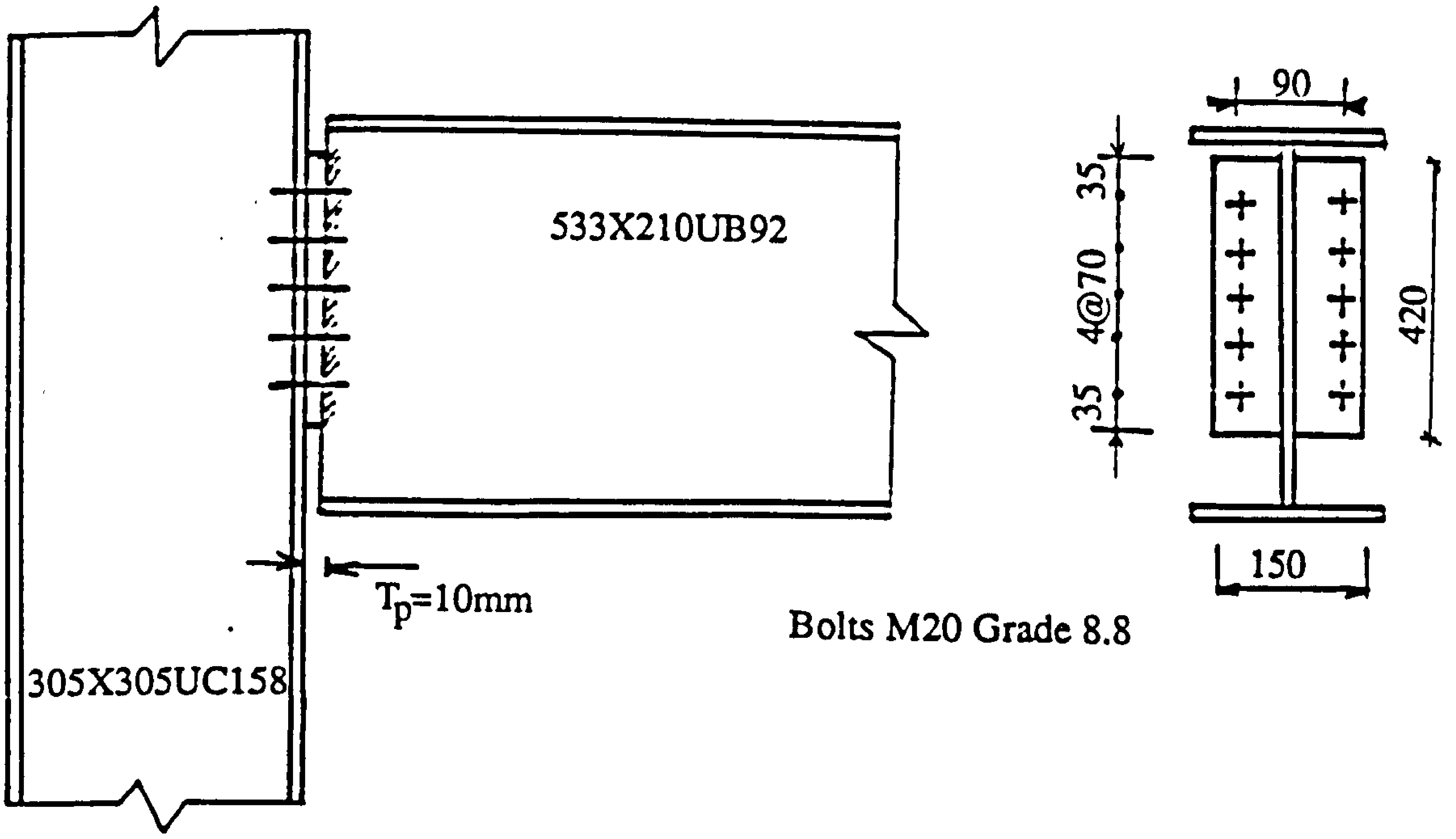


FIG.5.6a Connection details used in sub-frames SA1 and SB1.

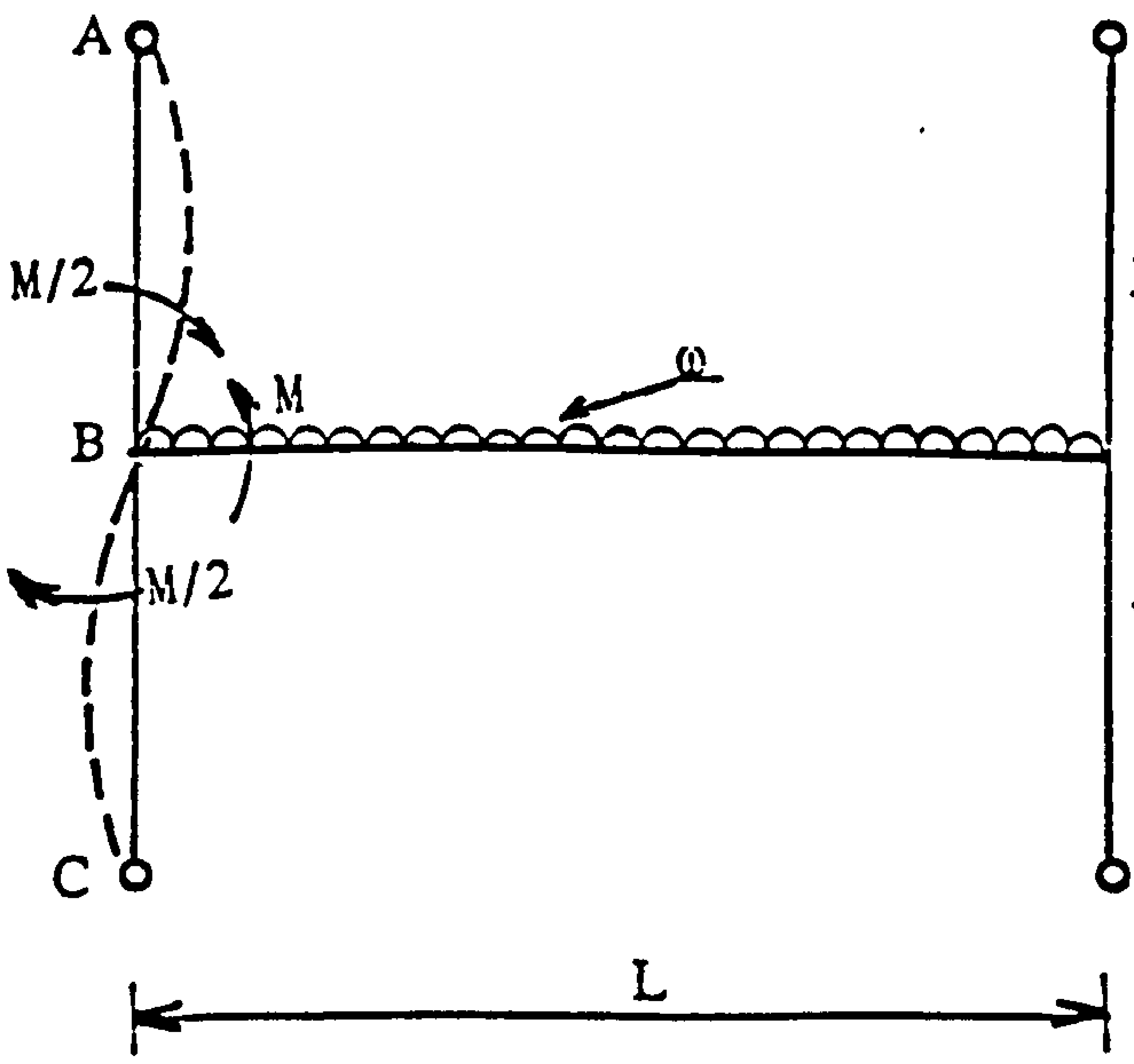


FIG.5-6b Sub-frames SA1.

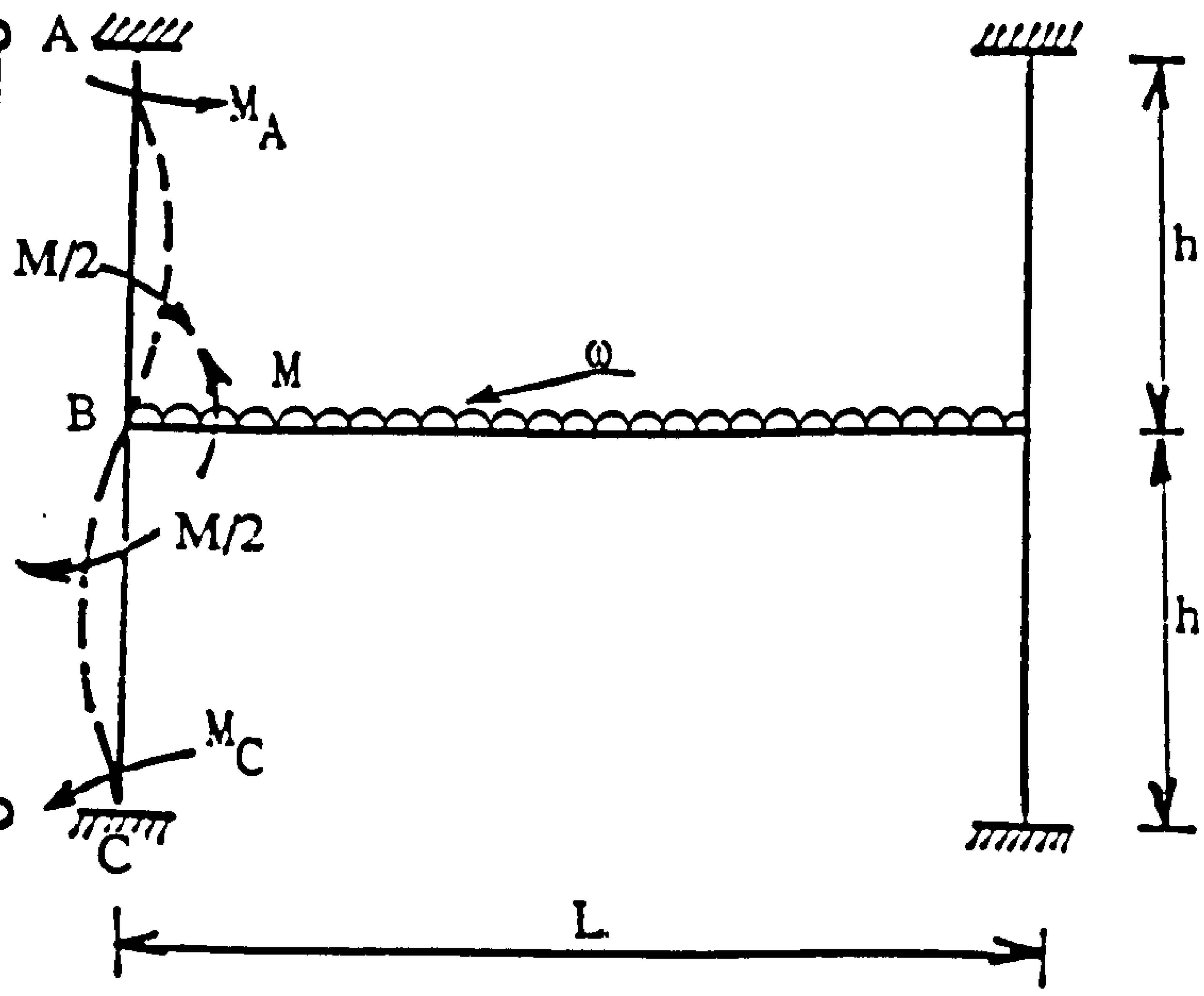


FIG.5-6c Sub-frame SB1.

FIG.5-6 Configuration of sub-frames SA1 and SB1 .

FIG.5-7 Analysis results using beam line method for sub-frame SA1.

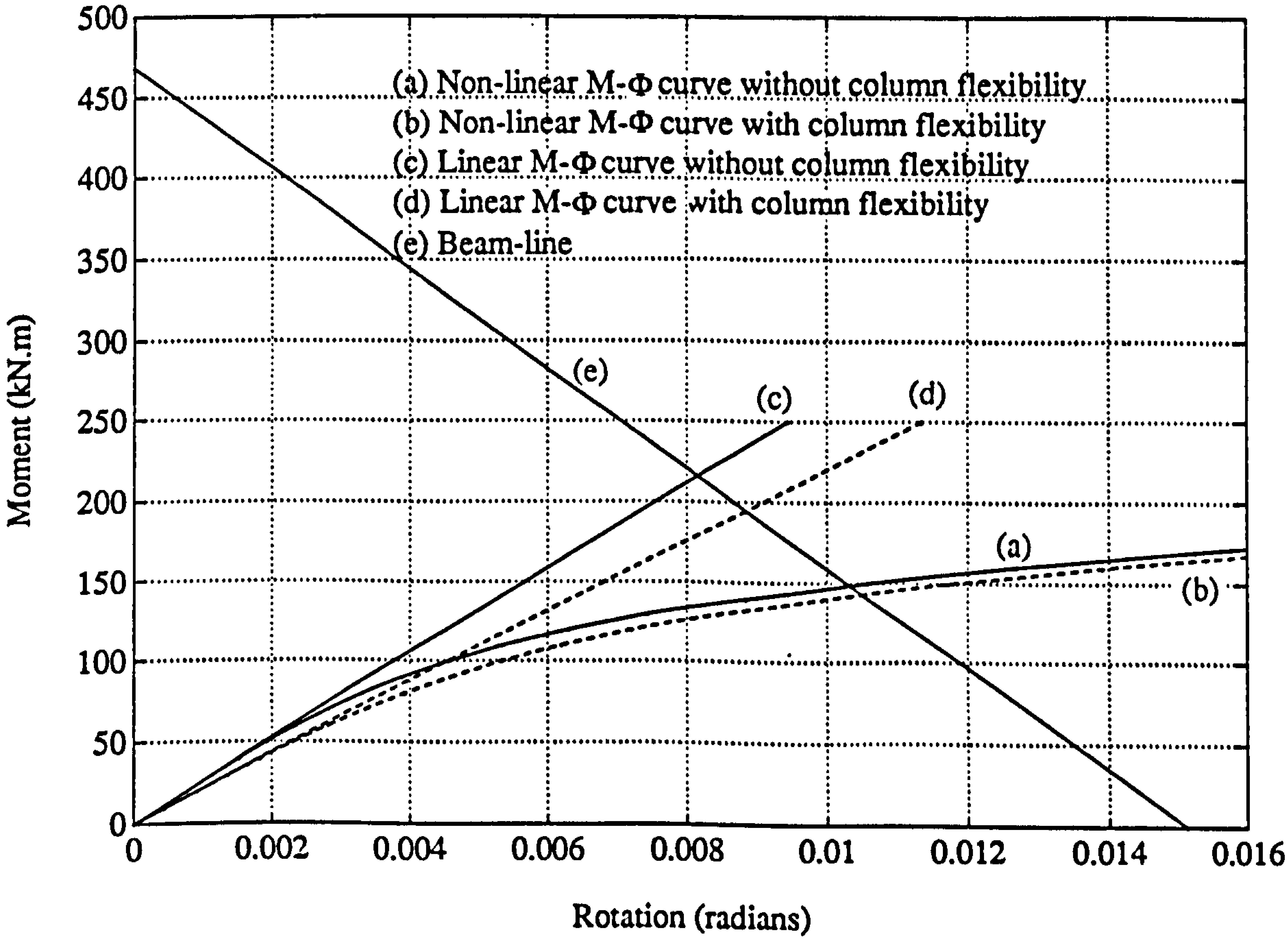


FIG.5-8 z/a versus x/a curves for sub-frame SA1
in terms of flexibilities

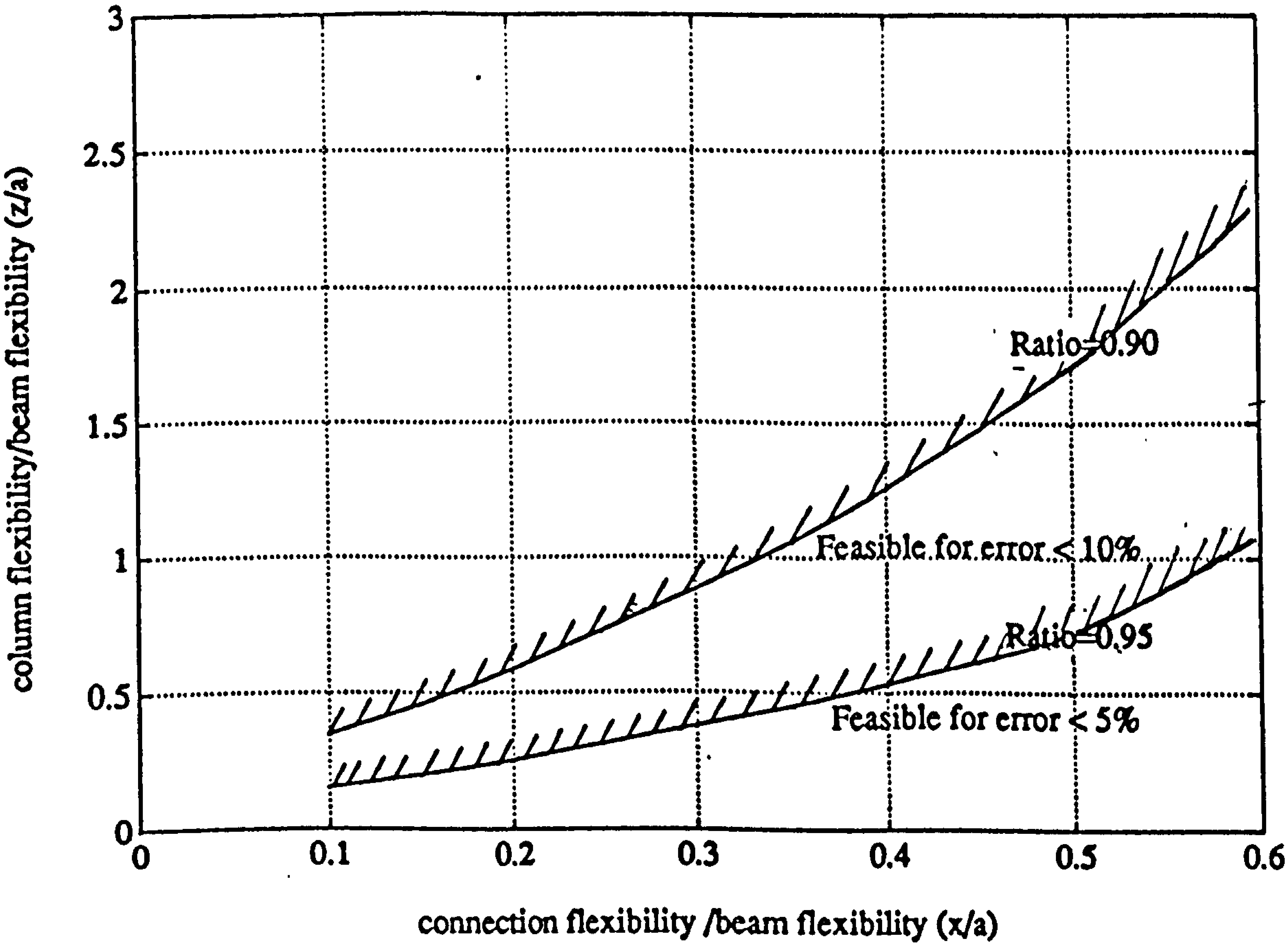


FIG.5-9 a/z versus a/x curves for sub-frame SA1
in terms of stiffnesses

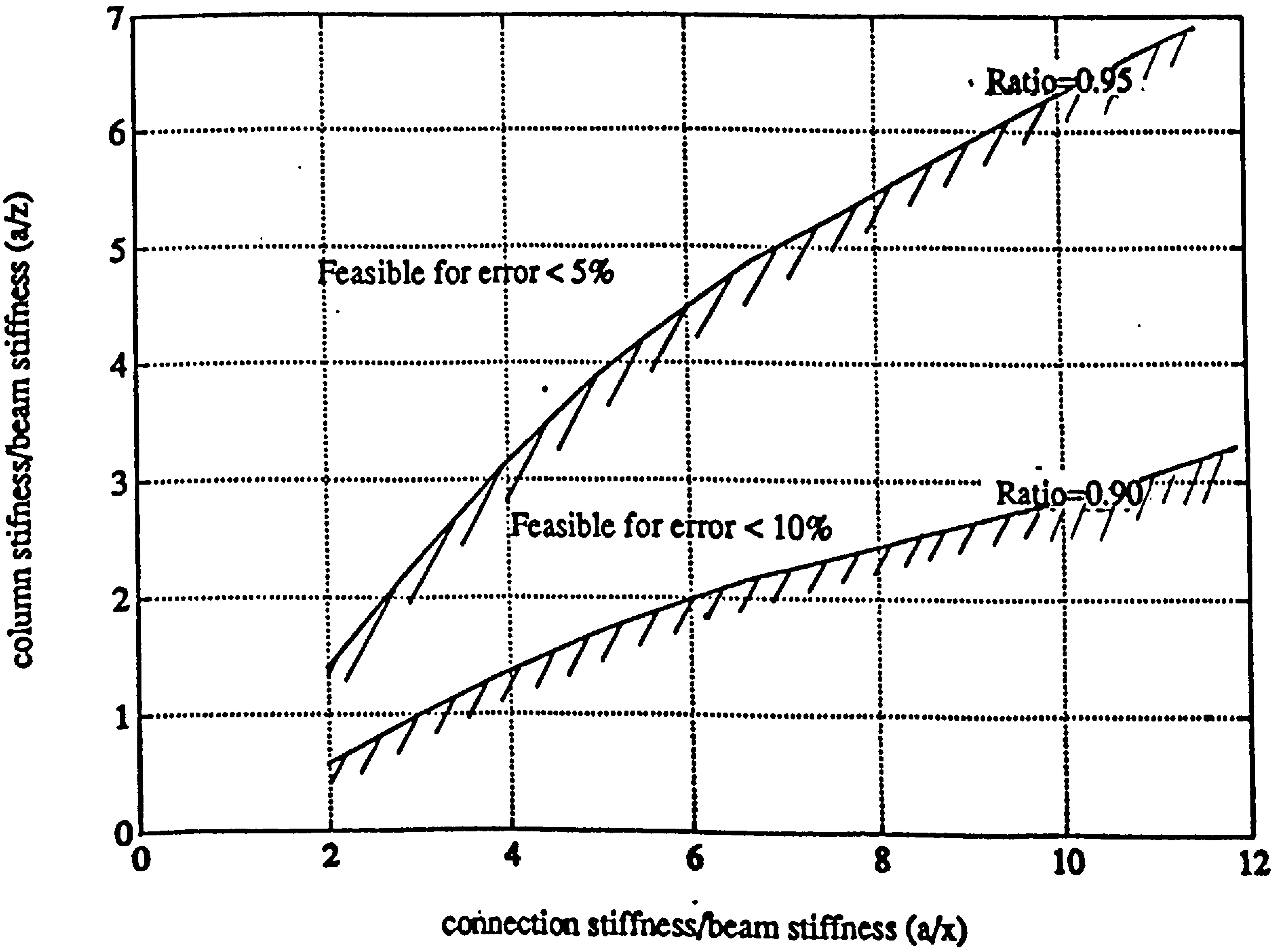


FIG.5-10 Analysis results using beam line method for sub-frame SB1.

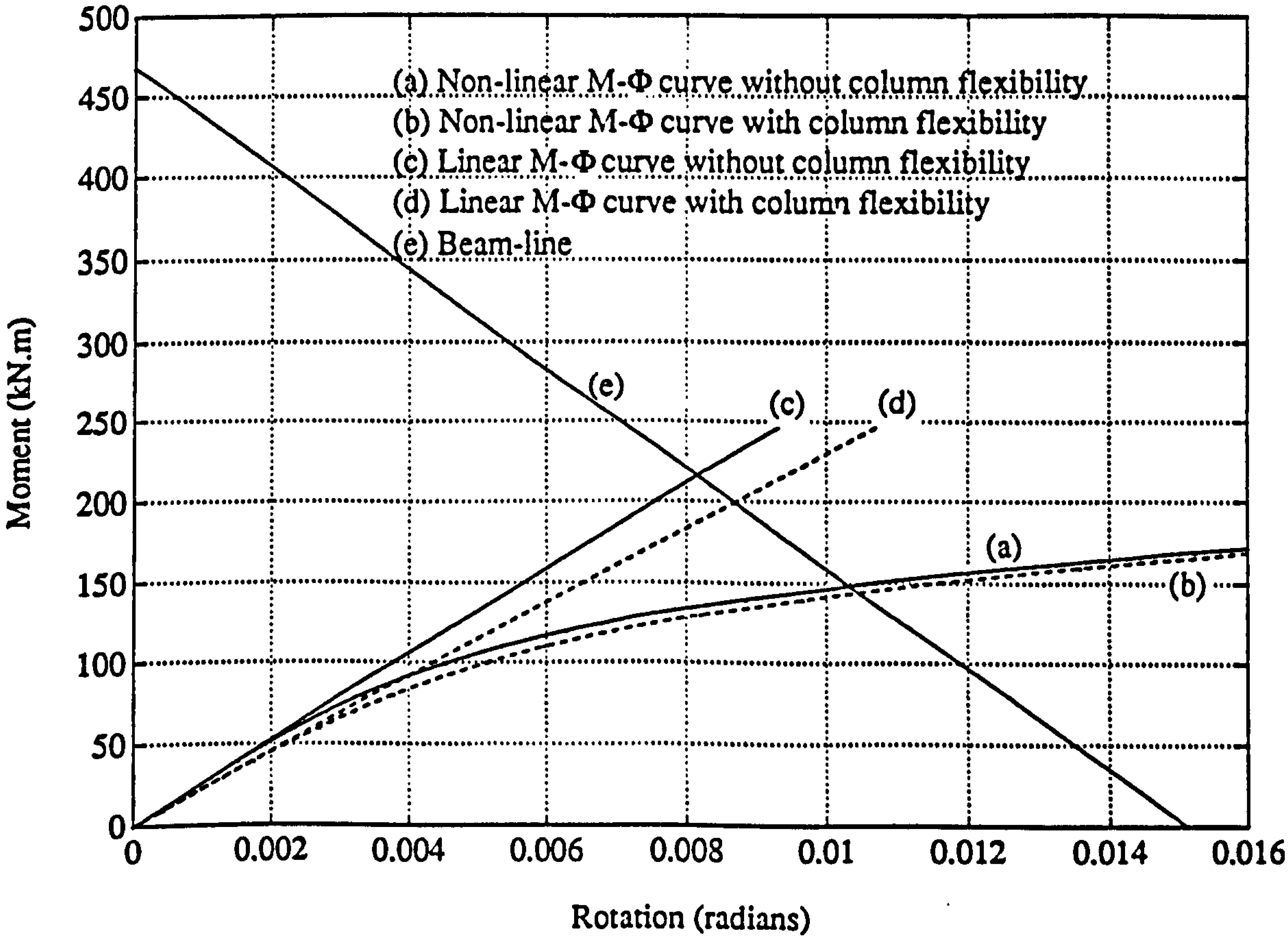


FIG.5-11 z/a versus x/a curves for sub-frame SB1
in terms of flexibilities

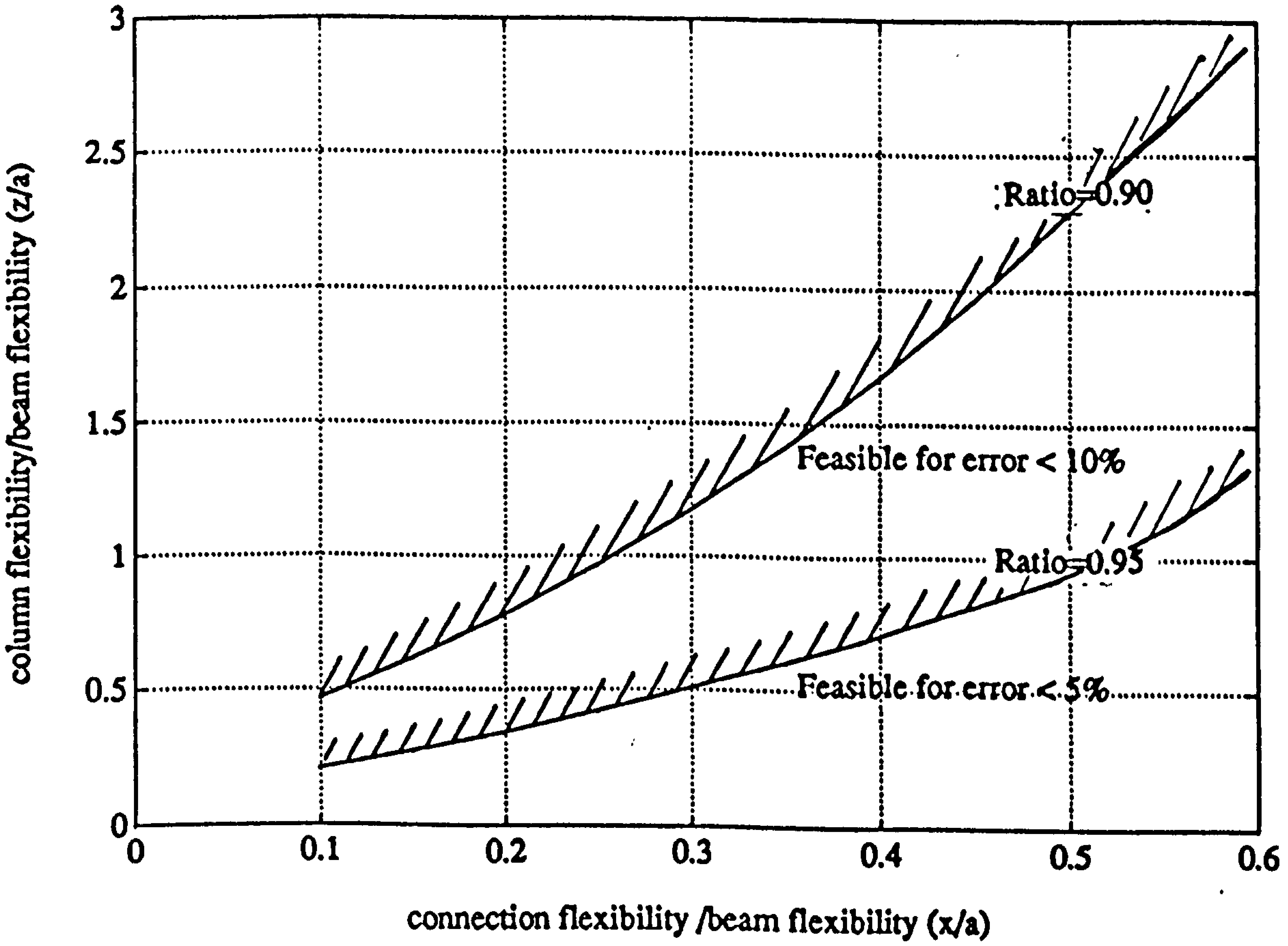
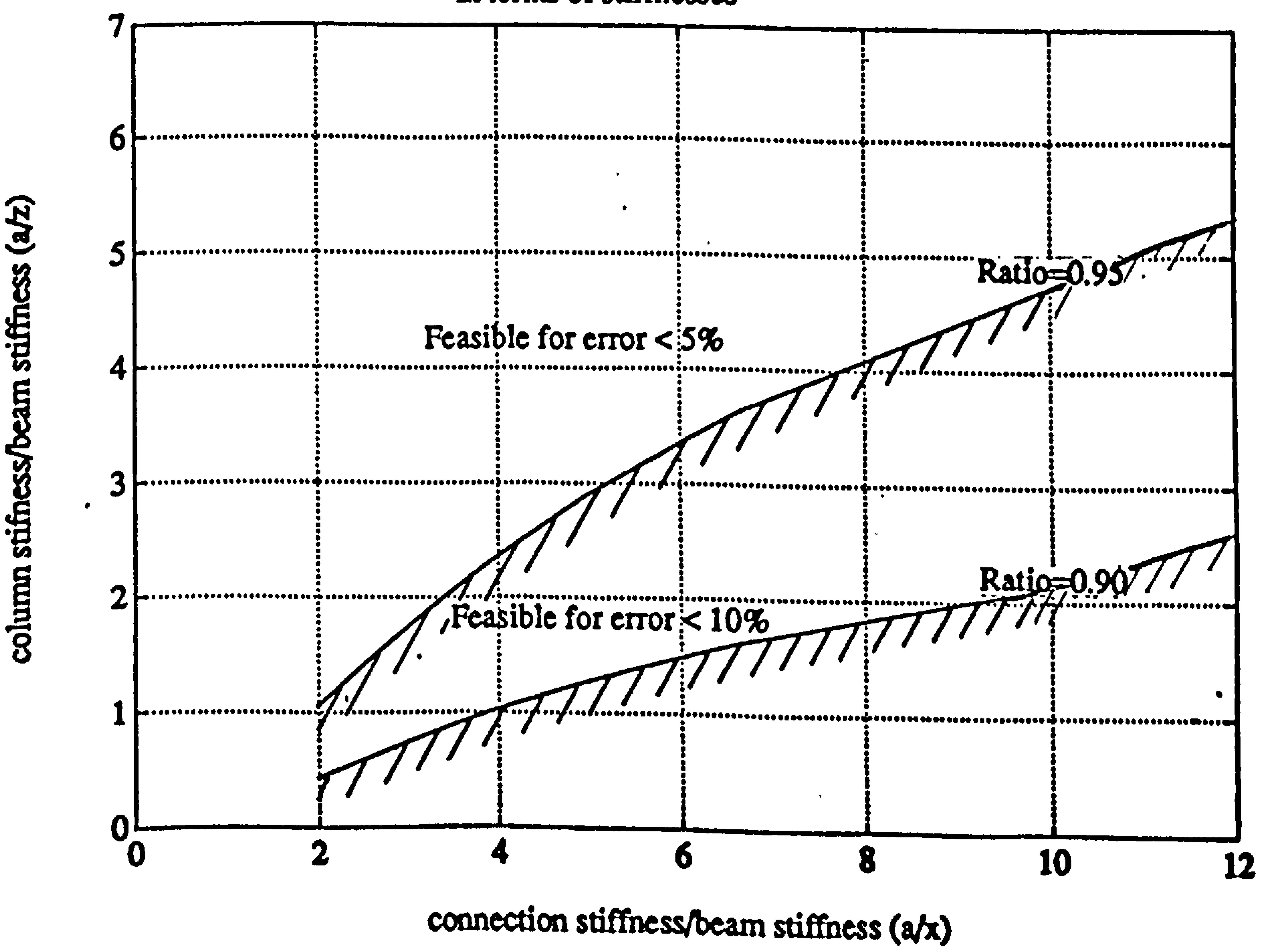


FIG.5-12 a/z versus a/x curves for sub-frame SB1
in terms of stiffnesses



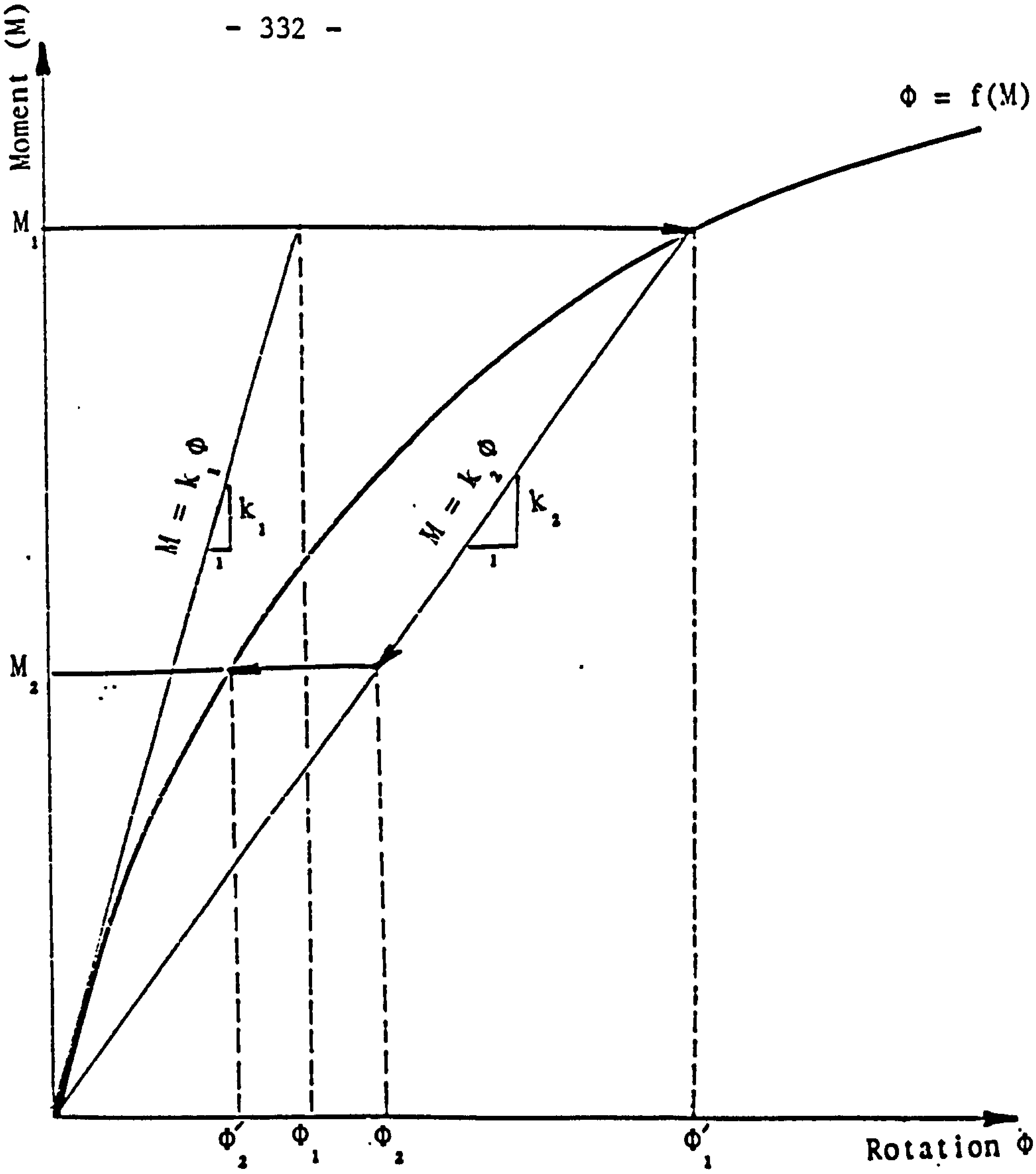


FIG.5-13 Iterative analysis procedure

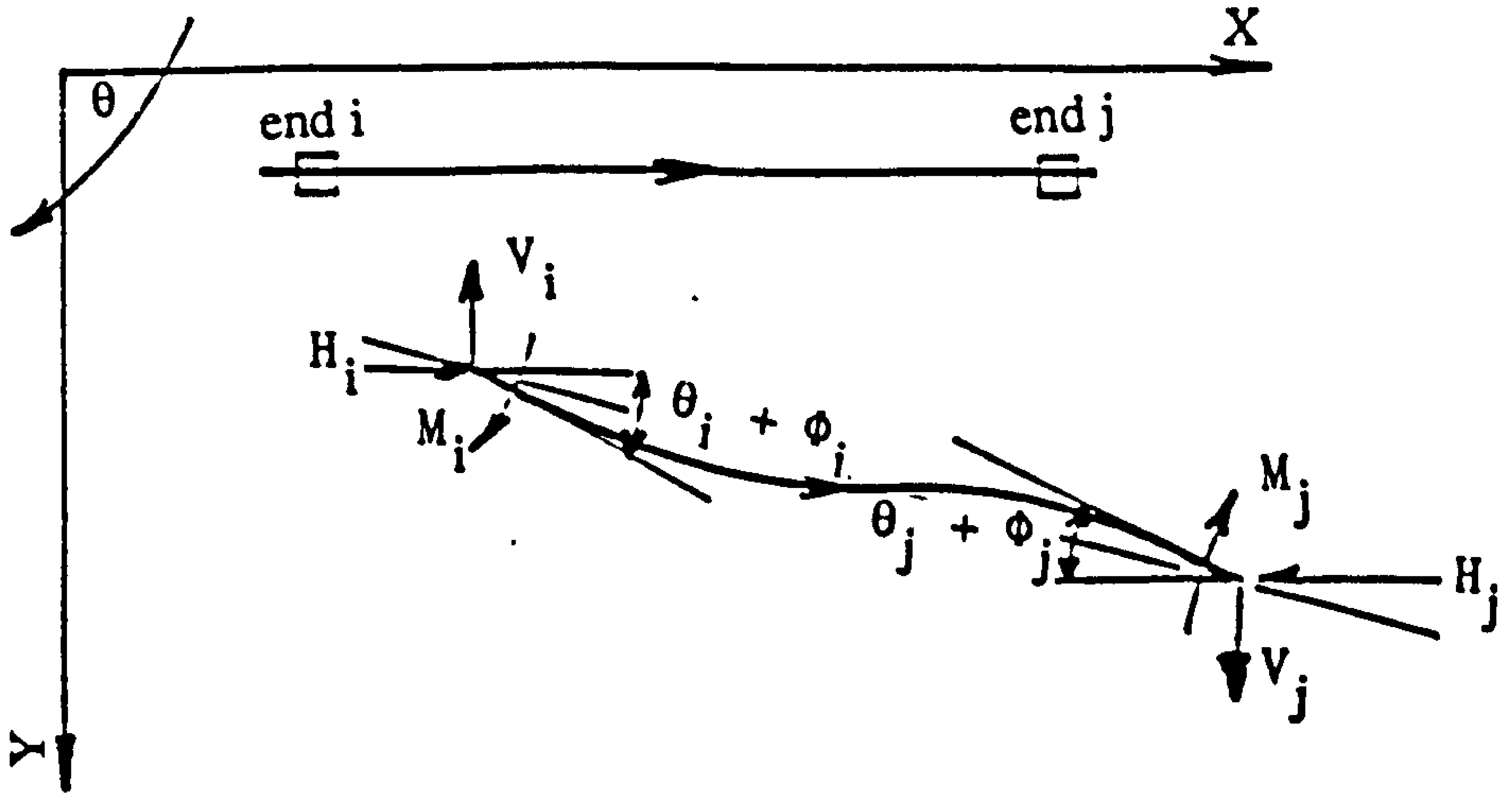


FIG.5-14 Member i-j with semi-rigid connections at its ends

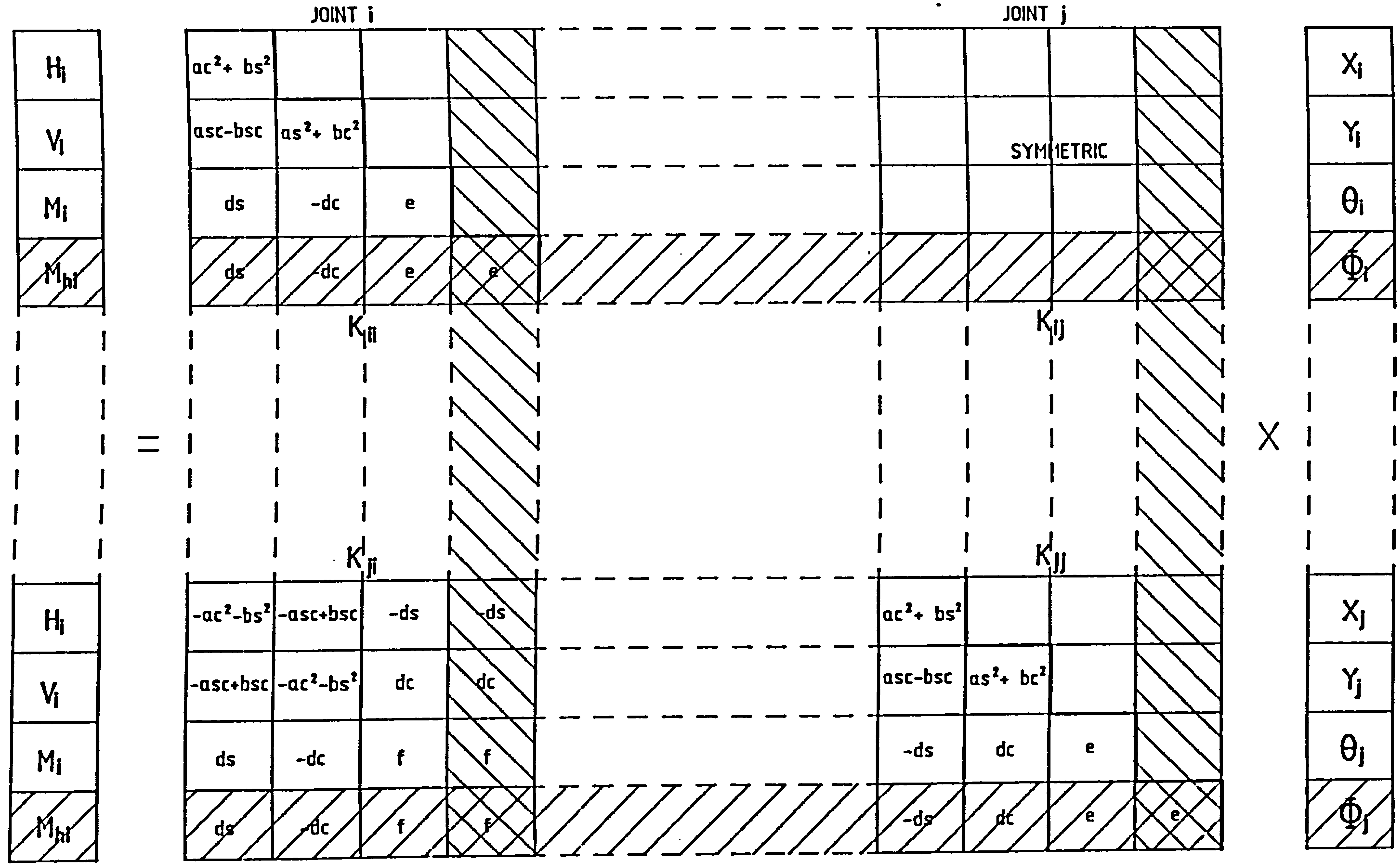


Figure 5.15 Contribution of semi-rigid jointed member i-j in the overall stiffness matrix.

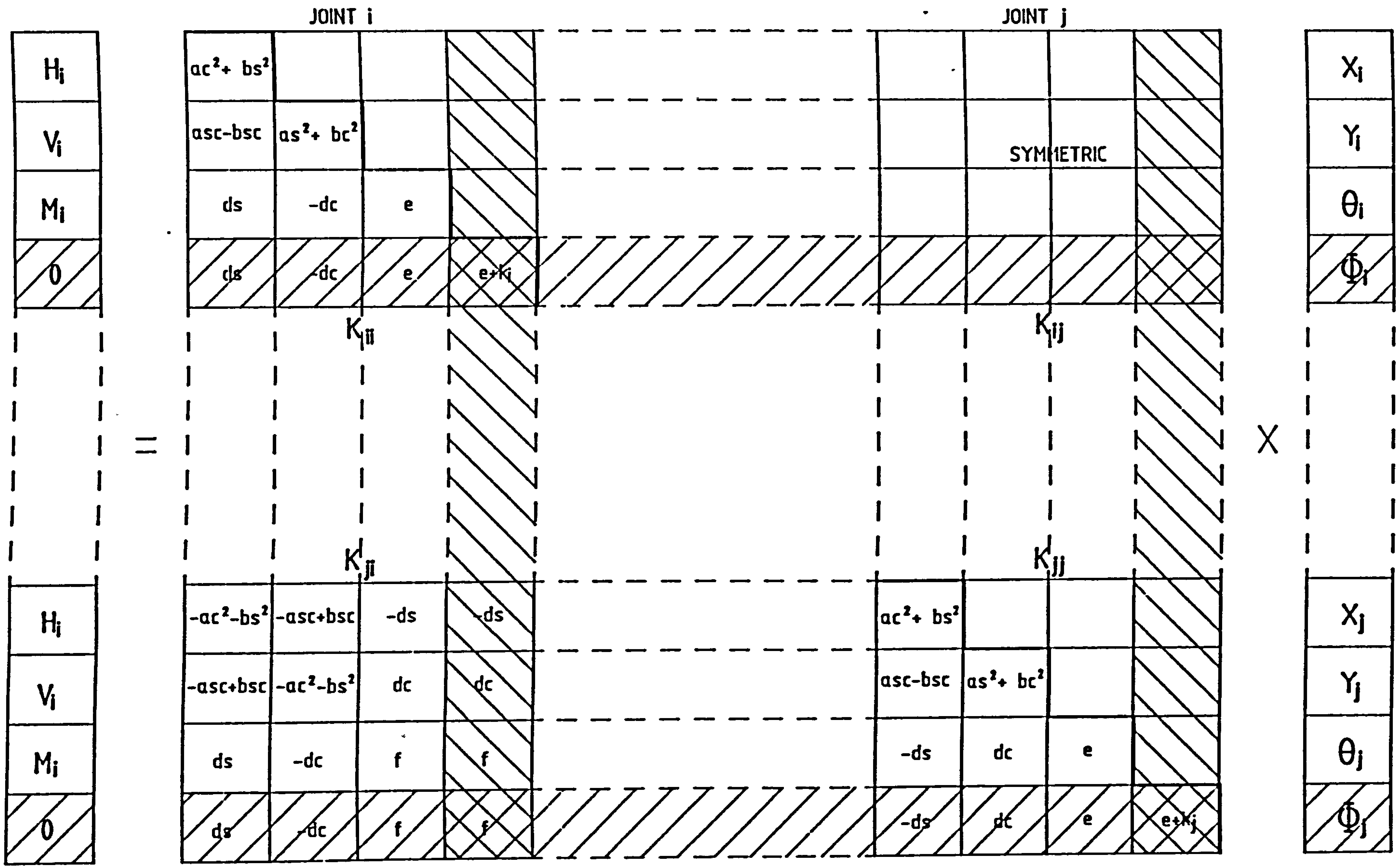
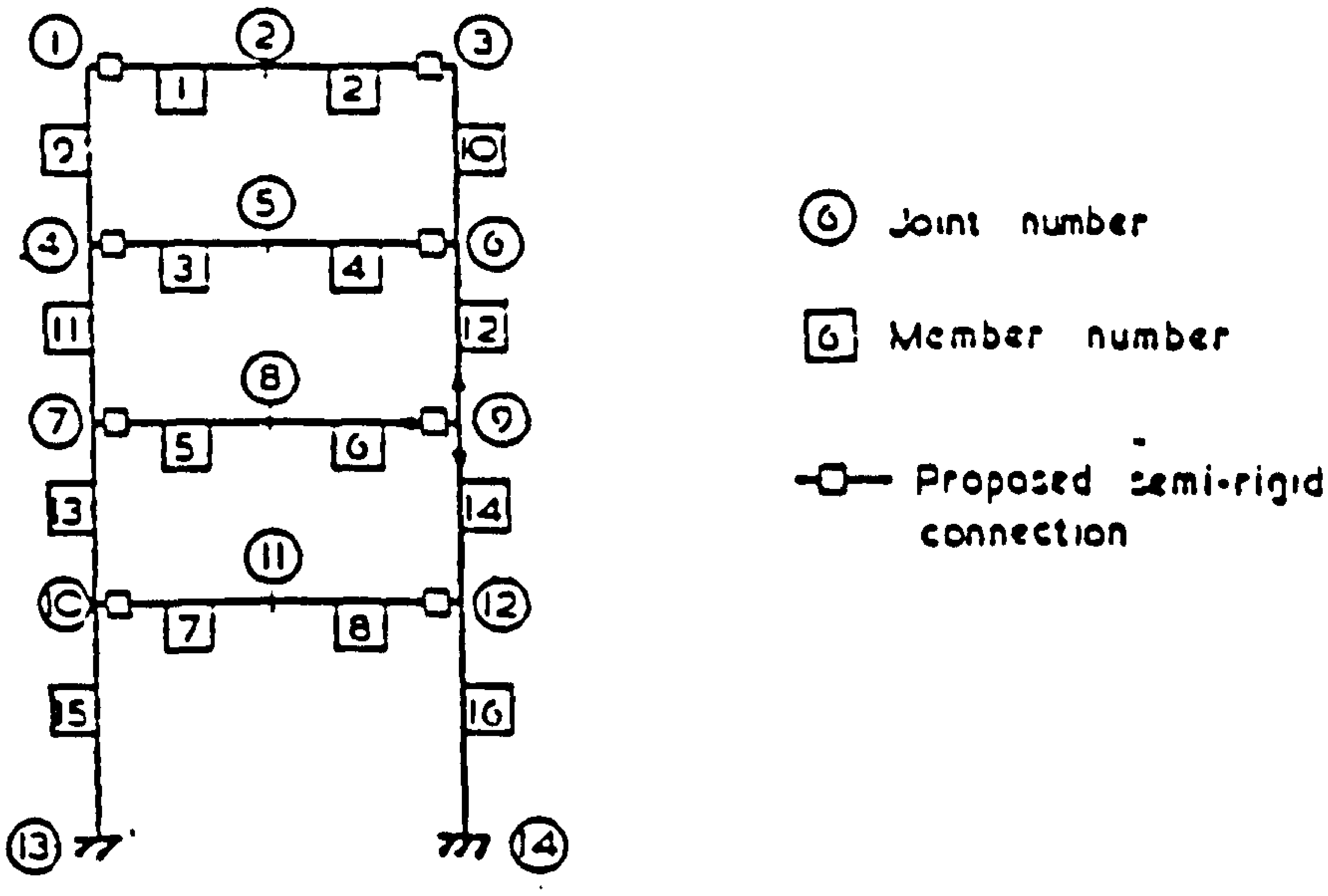
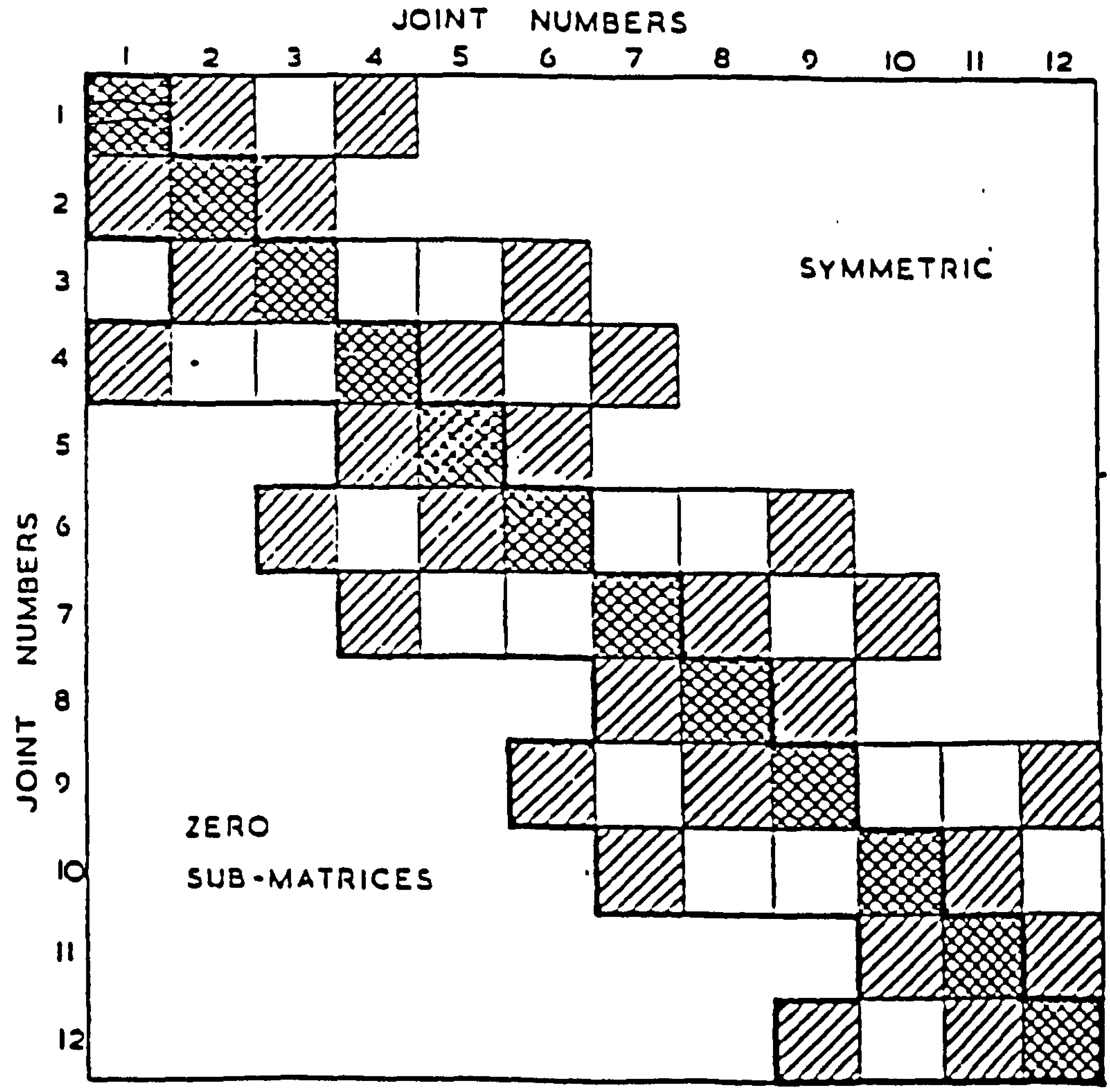


Figure 5.16 Contribution of semi-rigid jointed member i-j in the overall stiffness matrix in a re-arranged form.



a) Frame configuration.



b) Overall stiffness matrix.

FIG.5-17 Overall stiffness matrix of a rigid jointed three storey-one bay frame.

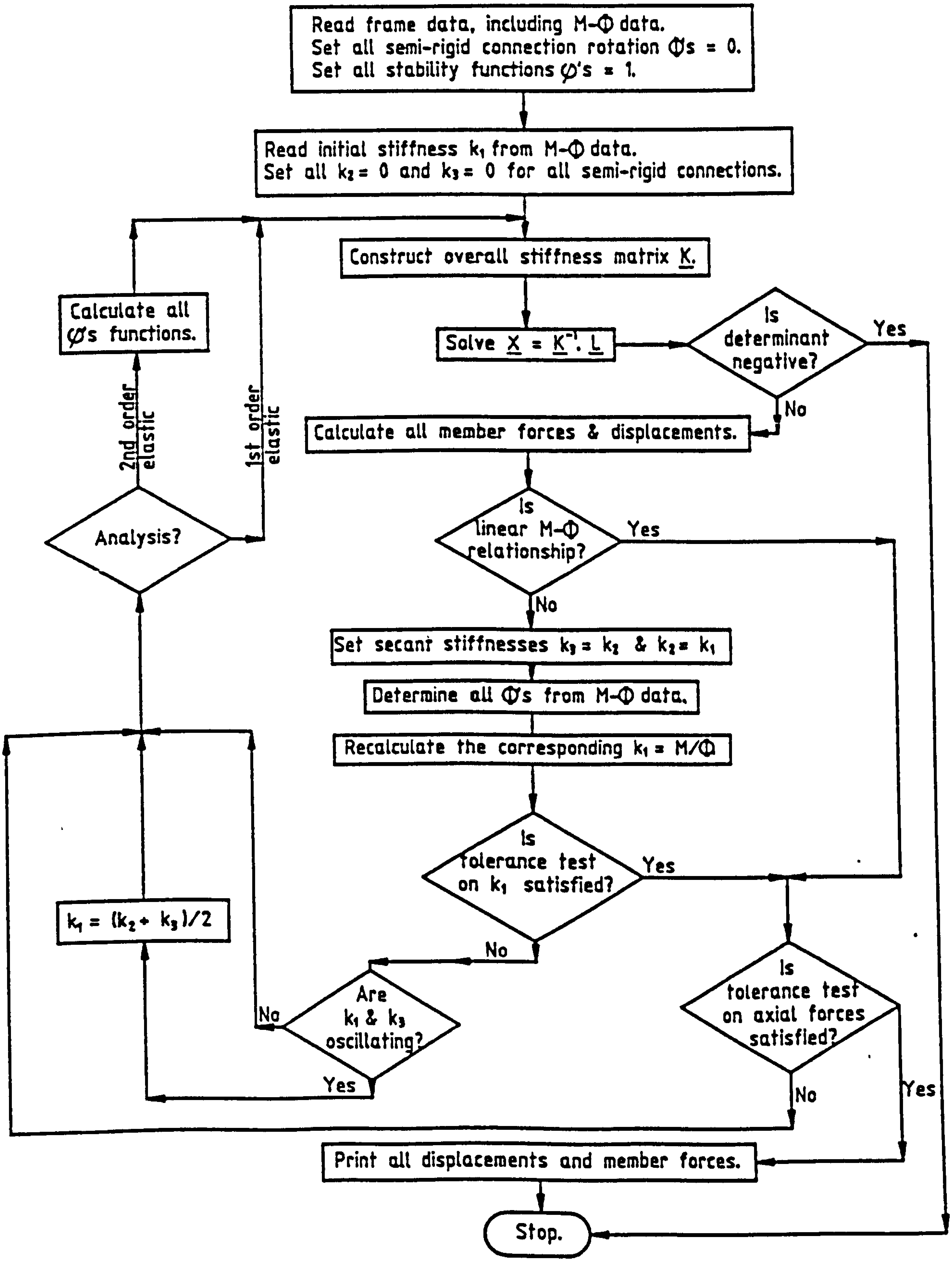


Figure 5.18 Flow diagram for analysis of semi-rigidly connected plane frames.

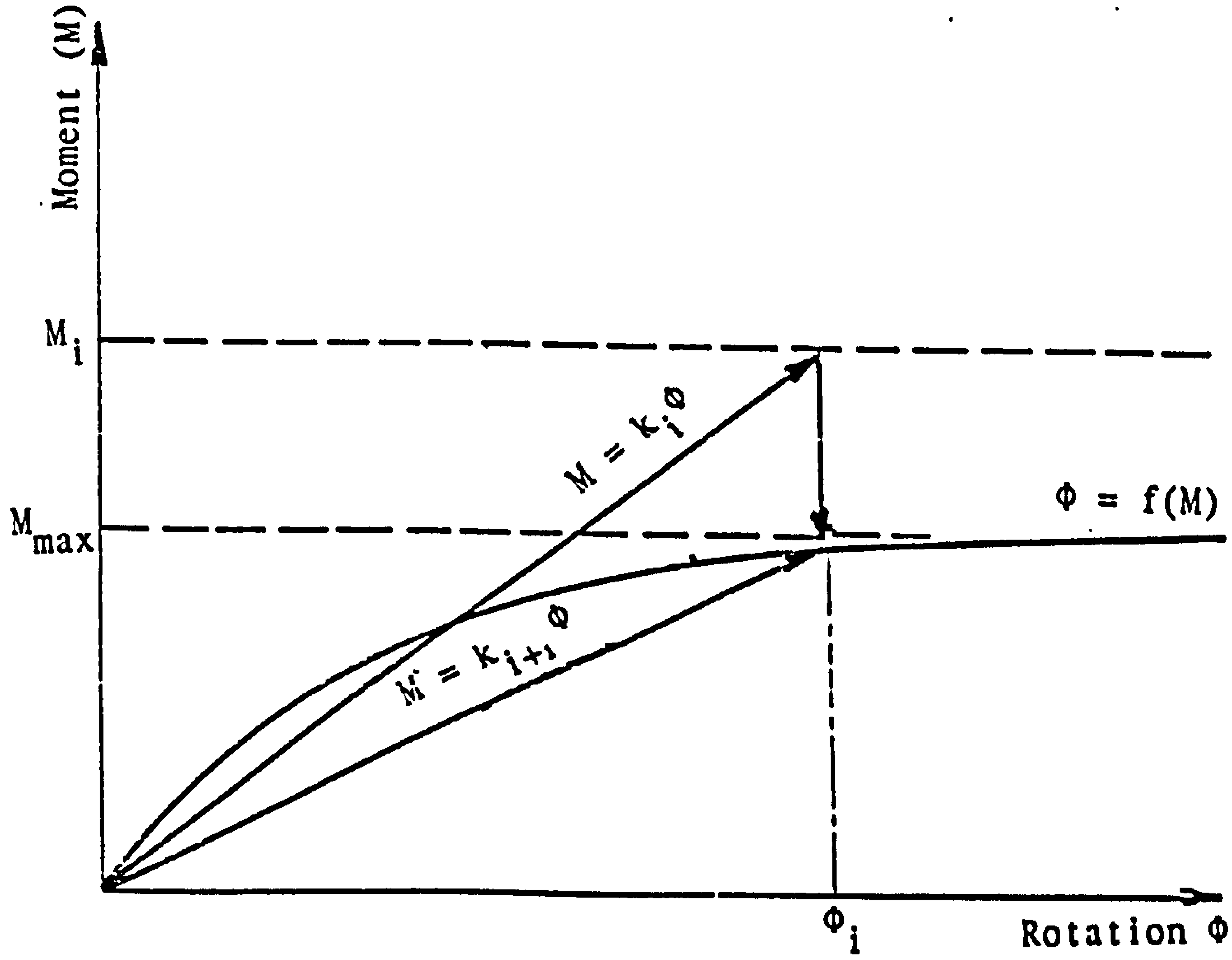
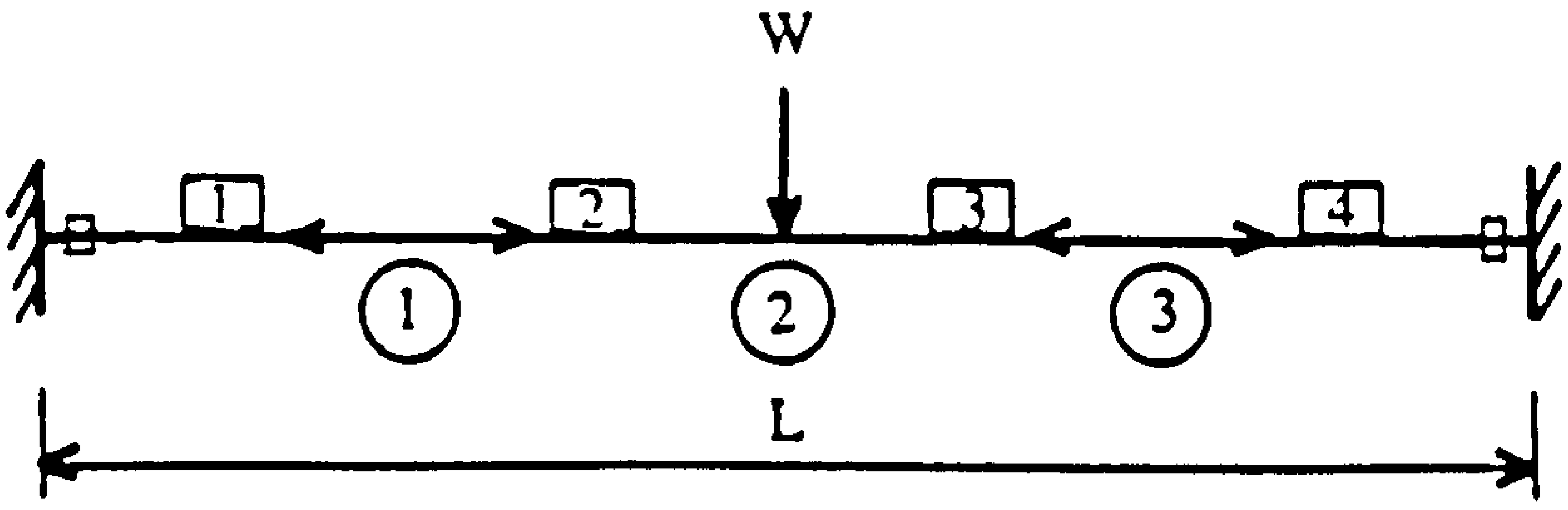


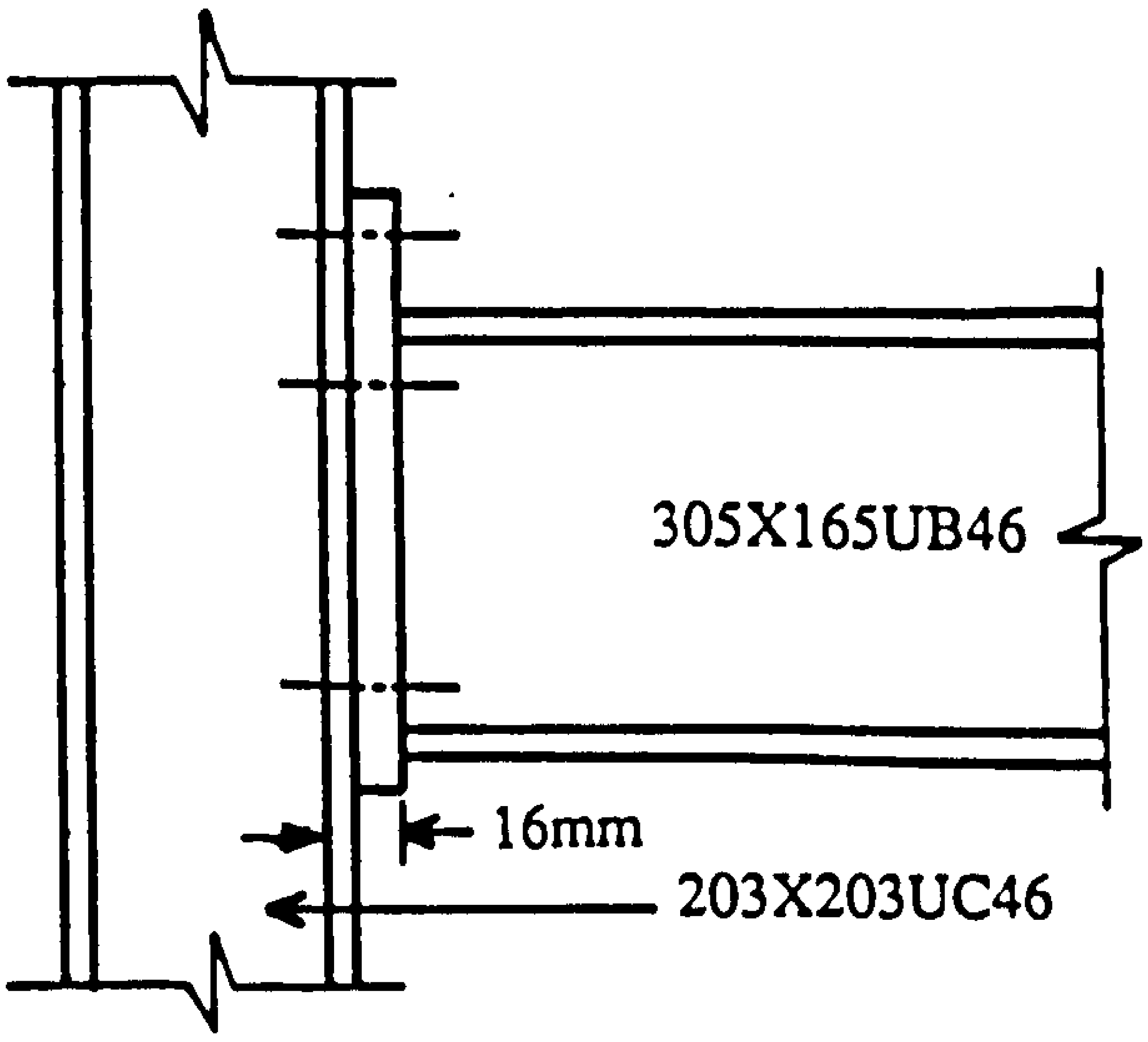
FIG.5-19 Moment out of range in the ith iteration



E=205kN/mm²

L=5000mm

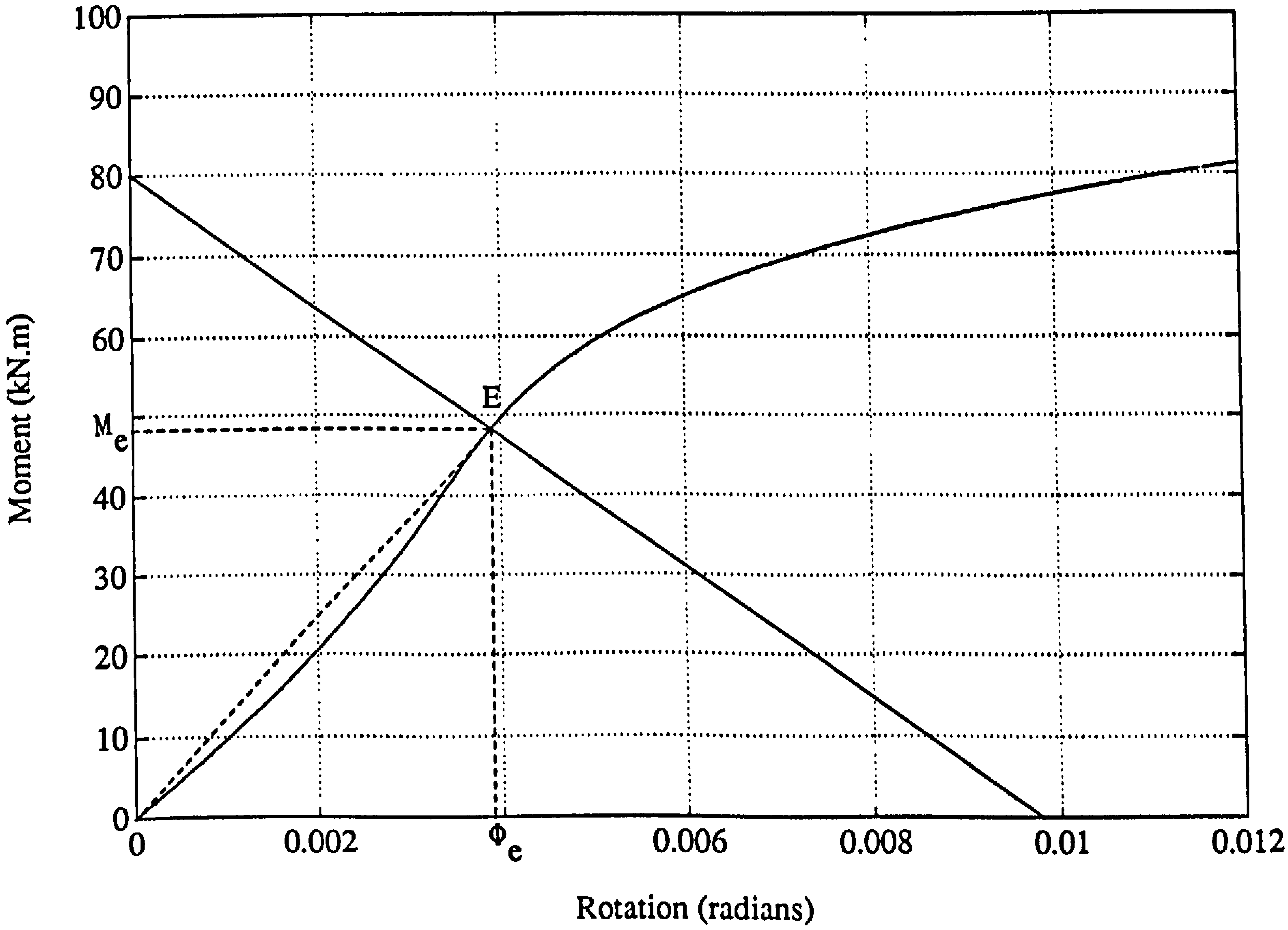
W=128kN

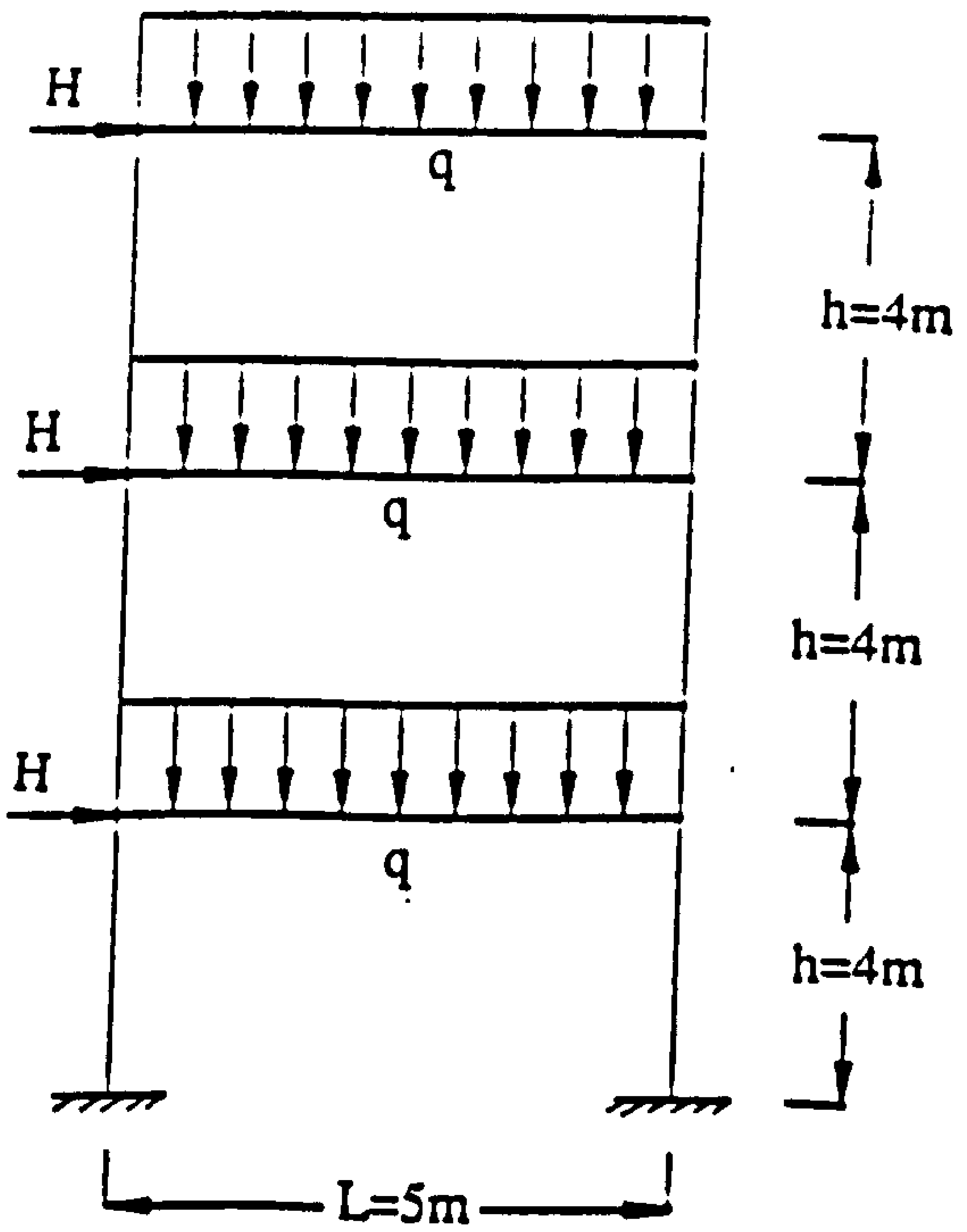


Iteration Number	M_e (kNm)	M_s (kNm)	y (mm)	$\Phi x10^{-3}$ (radians)	k (kNm/rads)
Semi-rigid analysis using initial secant stiffness					
1	43.28	116.72	9.71	4.500	9617.78
2	47.91	112.09	9.00	3.933	12181.54
3	48.19	111.81	8.96	3.898	12362.75
4	48.19	111.81	8.96	3.898	12362.75
Semi-rigid analysis using most flexible secant stiffness					
1	5.31	154.49	15.50	9.130	603.50
2	43.31	116.69	9.71	4.496	9633.01
3	47.91	112.09	9.00	3.933	12181.54
4	48.19	111.81	8.96	3.898	12362.75
5	48.19	111.81	8.96	3.898	12362.75
Rigid analysis					
1	80.00	80.00	4.09	-	-

Figure 5.20 Numerical example

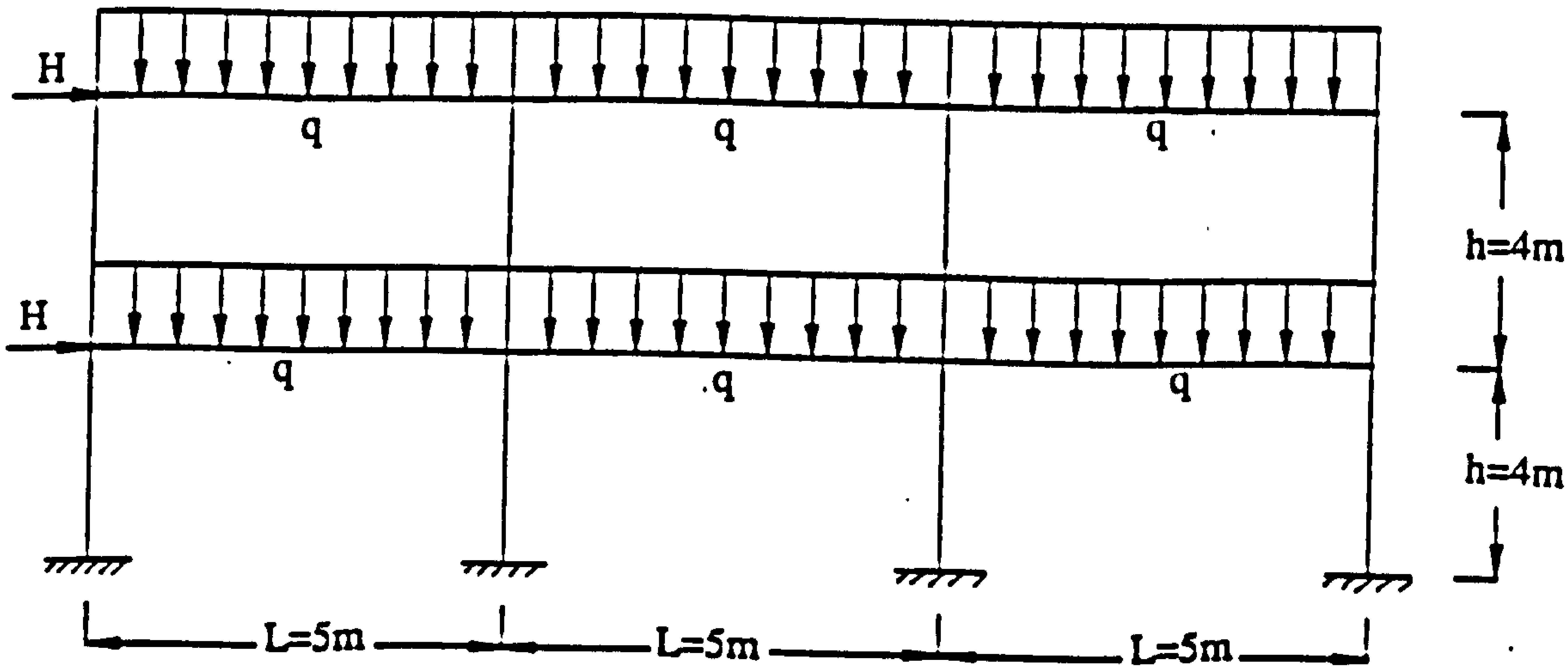
FIG.5-21 Analysis results using beam line method for the numerical example





(a) Frame A

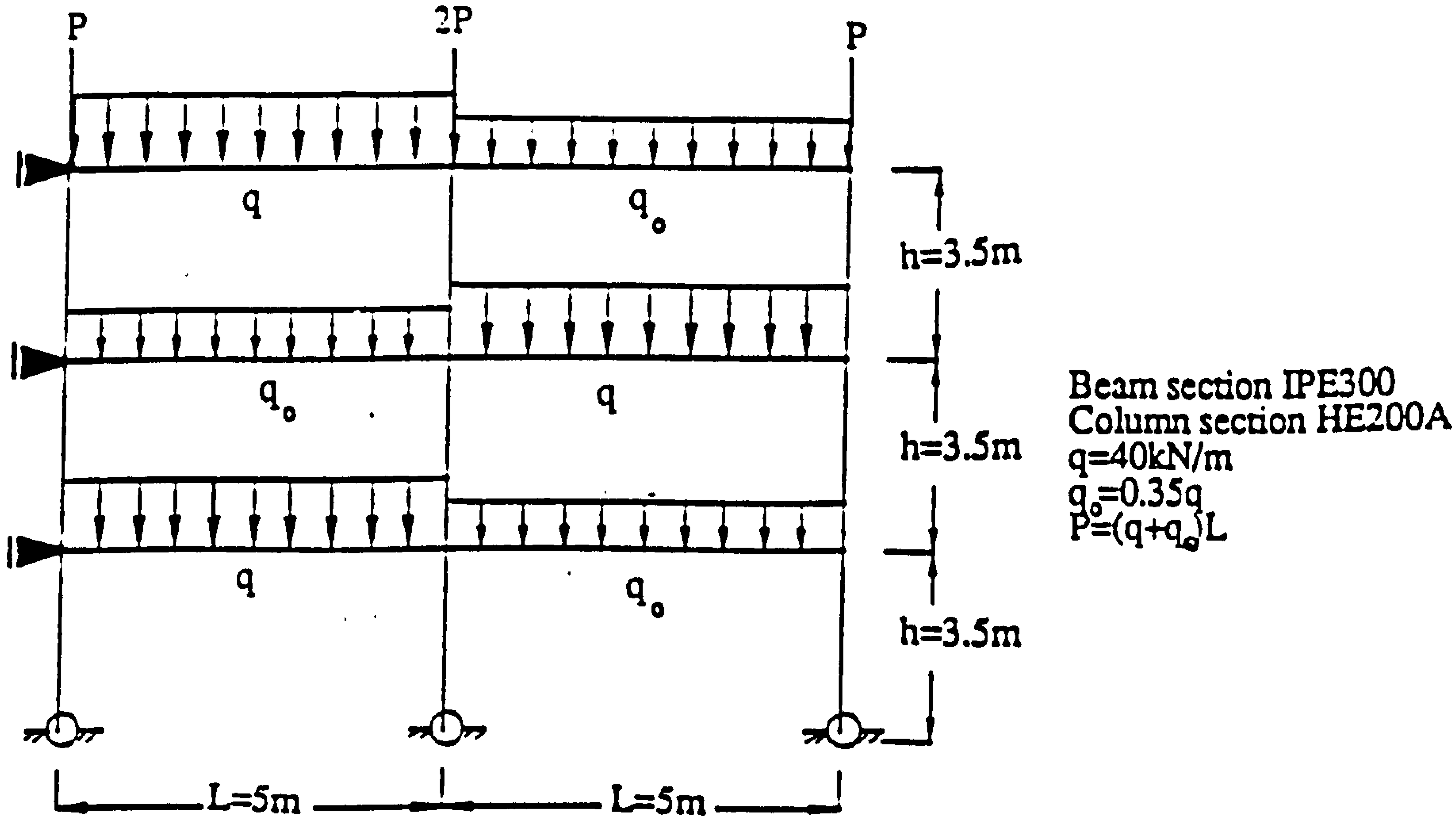
Beam section IPE300
Column section HE200B
q=40kN/m
H=0.05qL



(b) Frame B

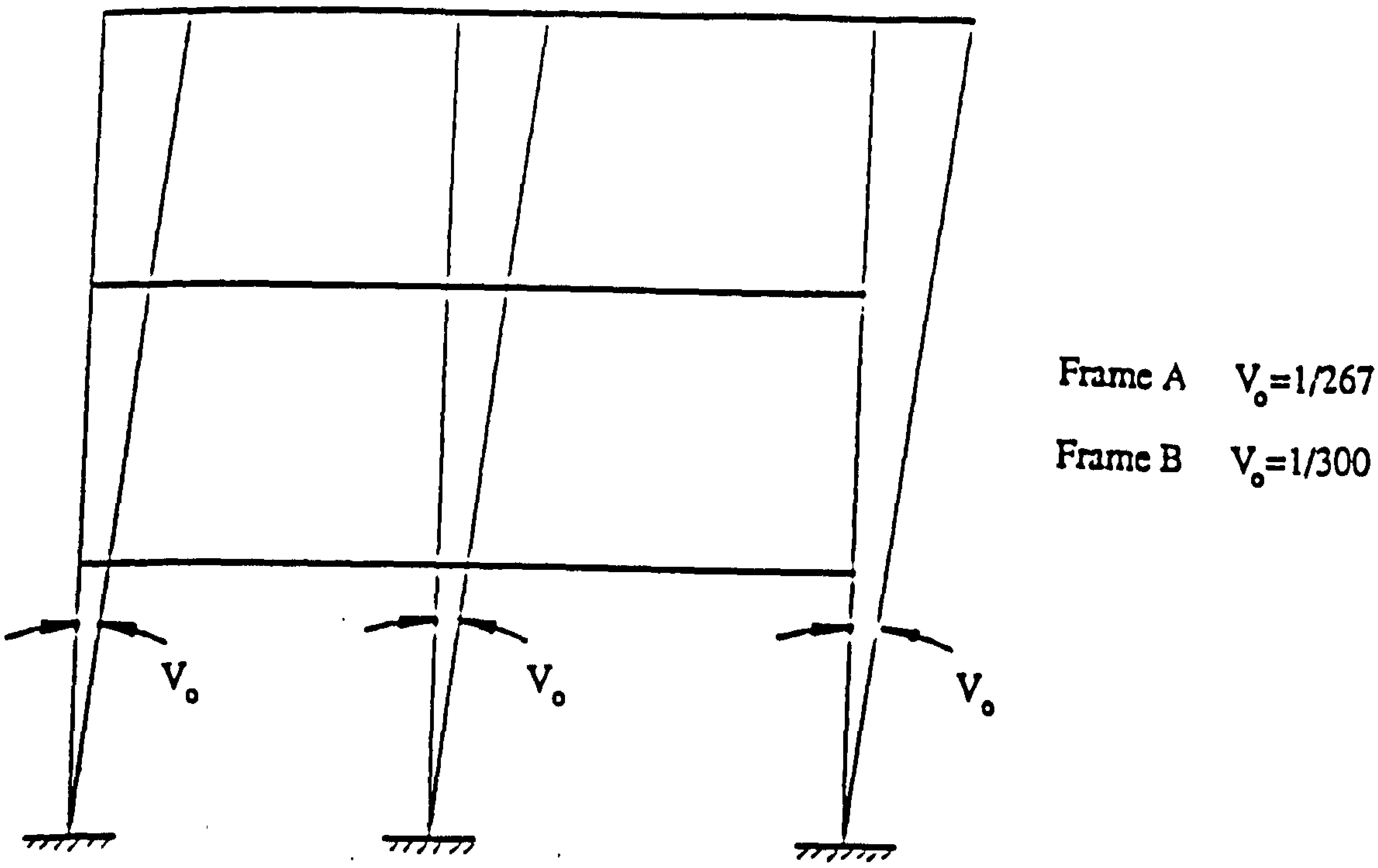
Beam section IPE300
Column section HE160A
q=40kN/m
H=0.025qL

FIG.5-22 Frame geometries and loading conditions for Frames A, B and C

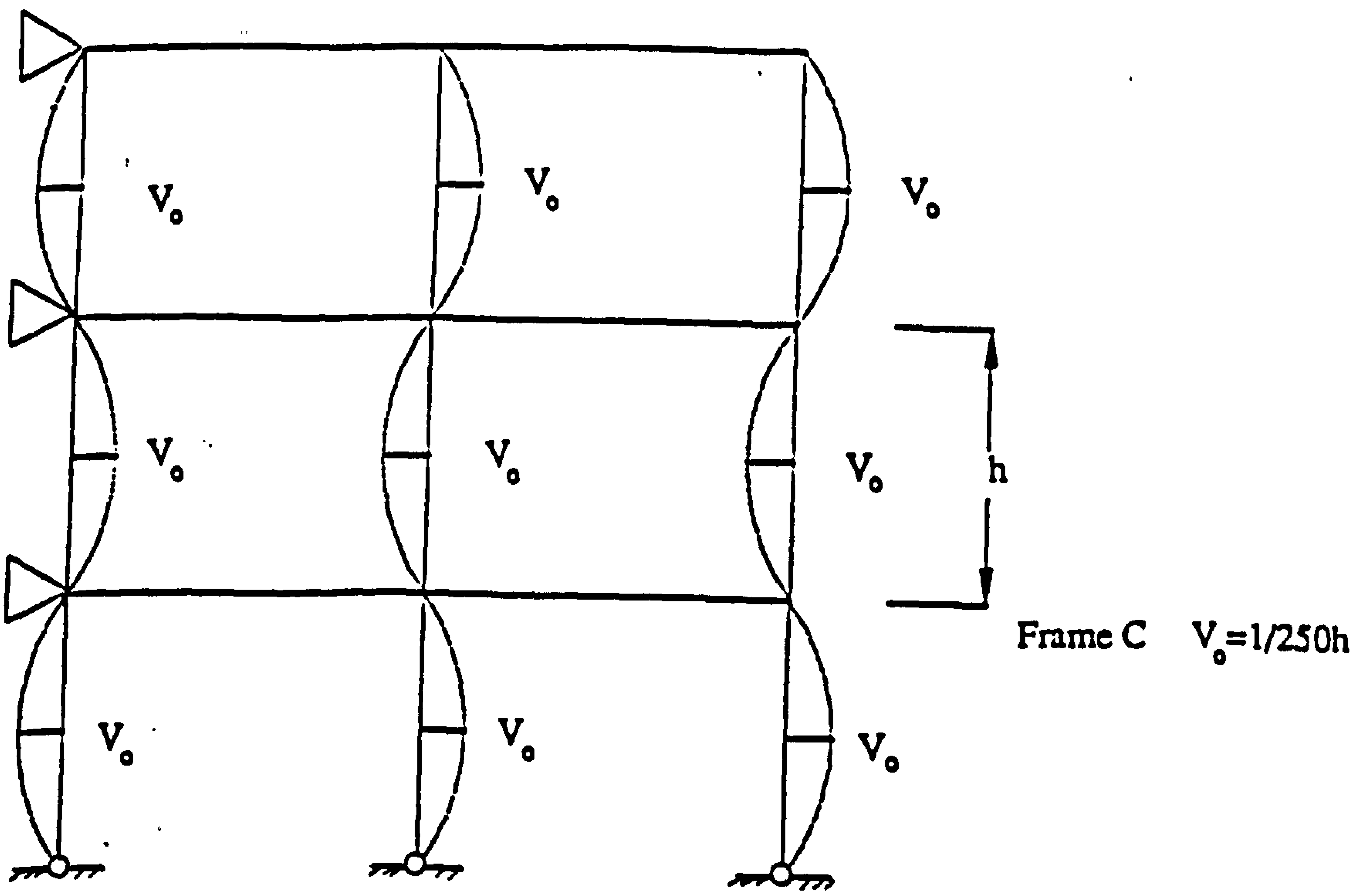


(c) Frame C

FIG.5-22 Continued

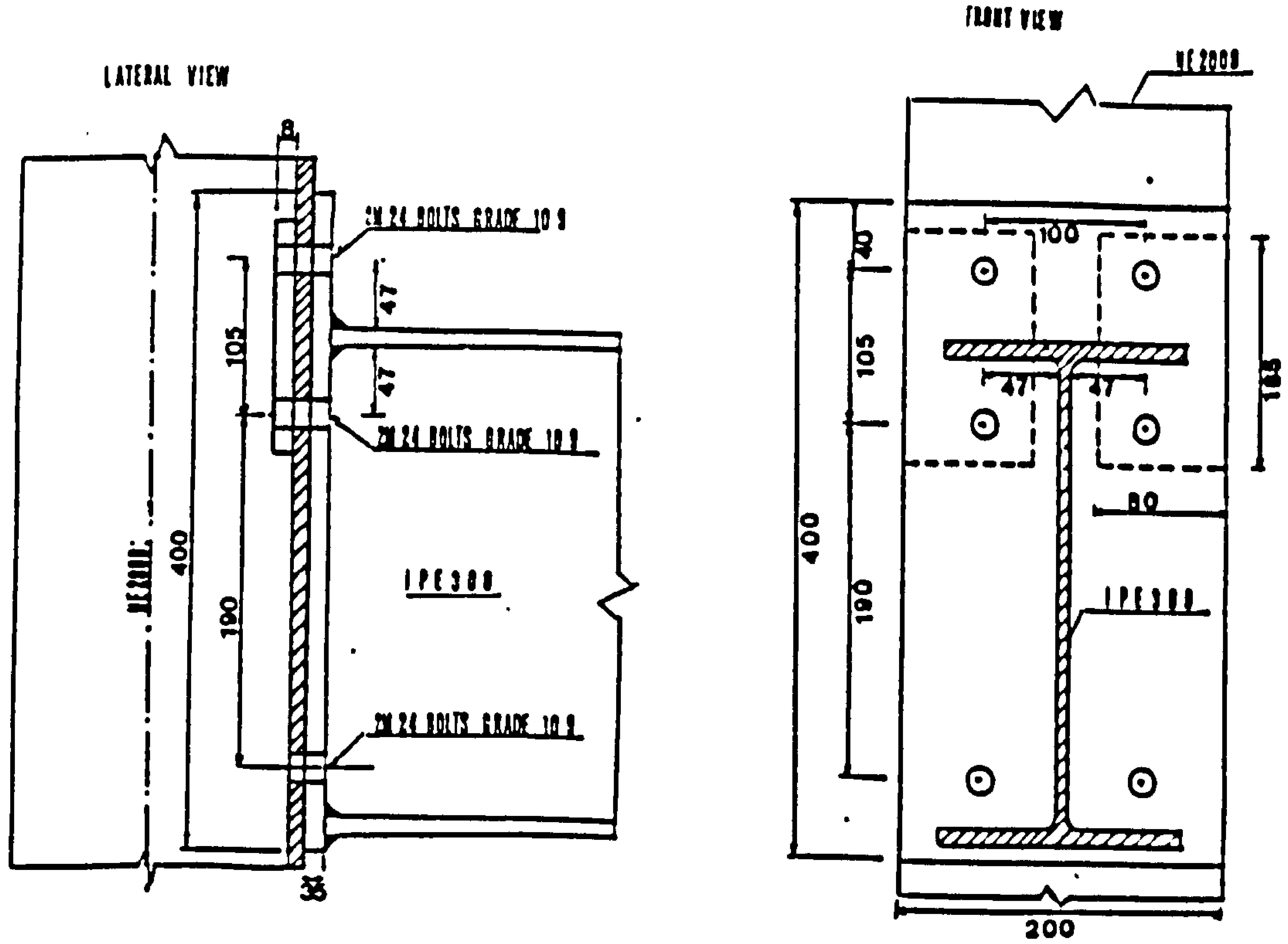


(a) Unbraced frames (Frames A and B)

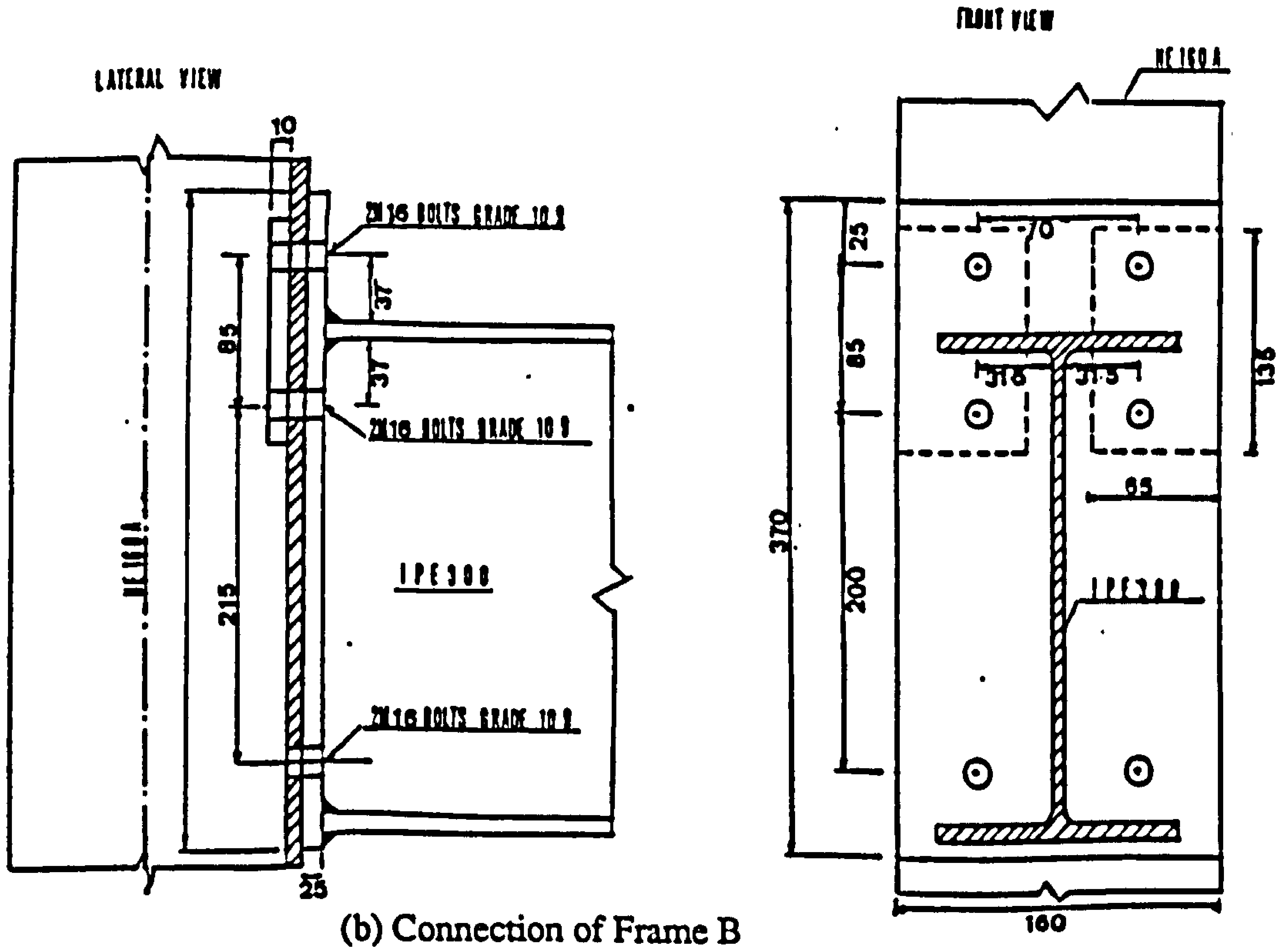


(b) Braced frames (Frame C)

FIG.5-23 Initial imperfections of braced and unbraced frames

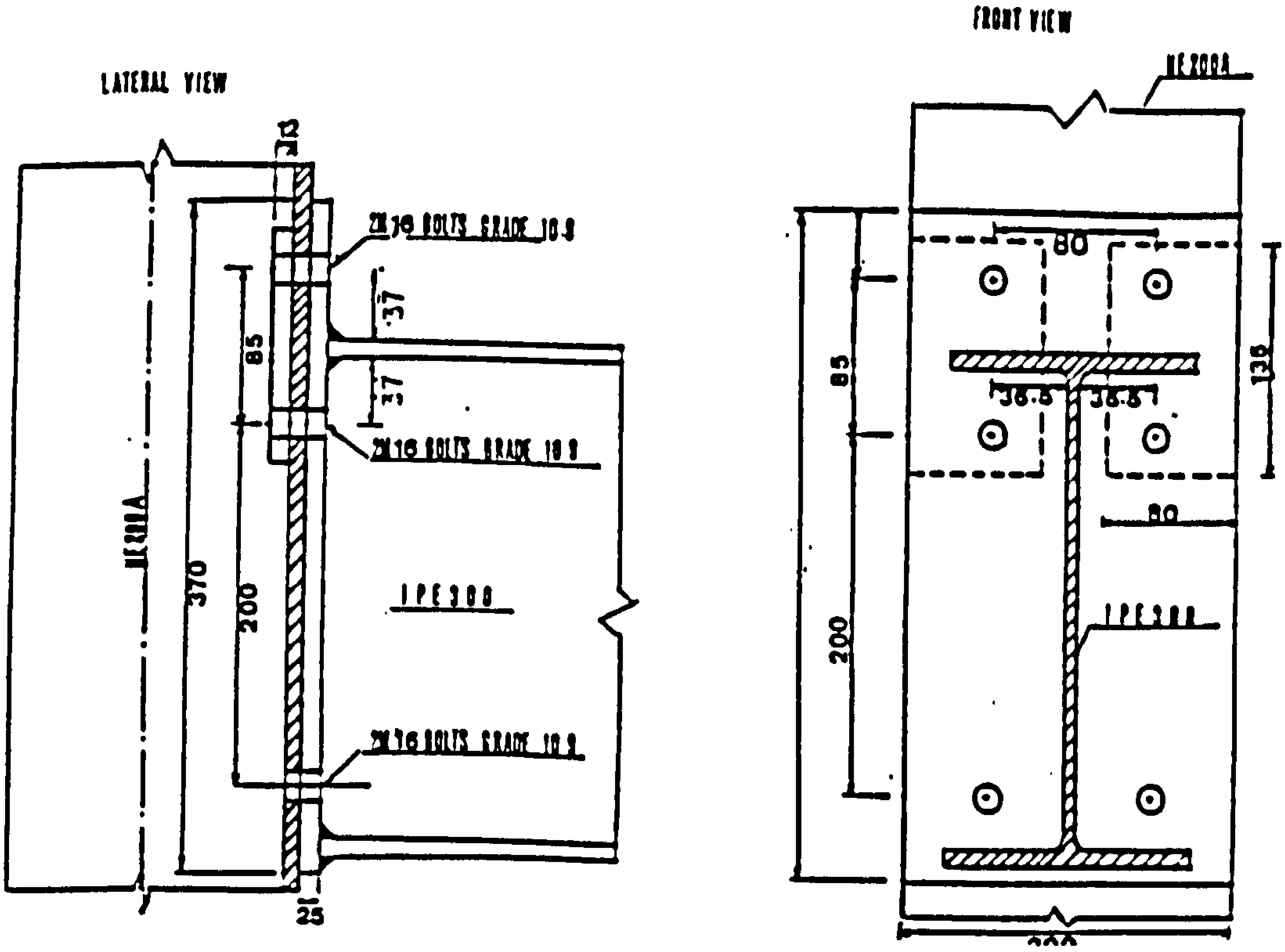


(a) Connection of Frame A



(b) Connection of Frame B

FIG.5-24 Connection details for Frames A ,B and C



(c) Connection of Frame C

FIG.5-24 Continued

FIG.5-25 Moment-rotation curves of the joint of Frames A, B and C.

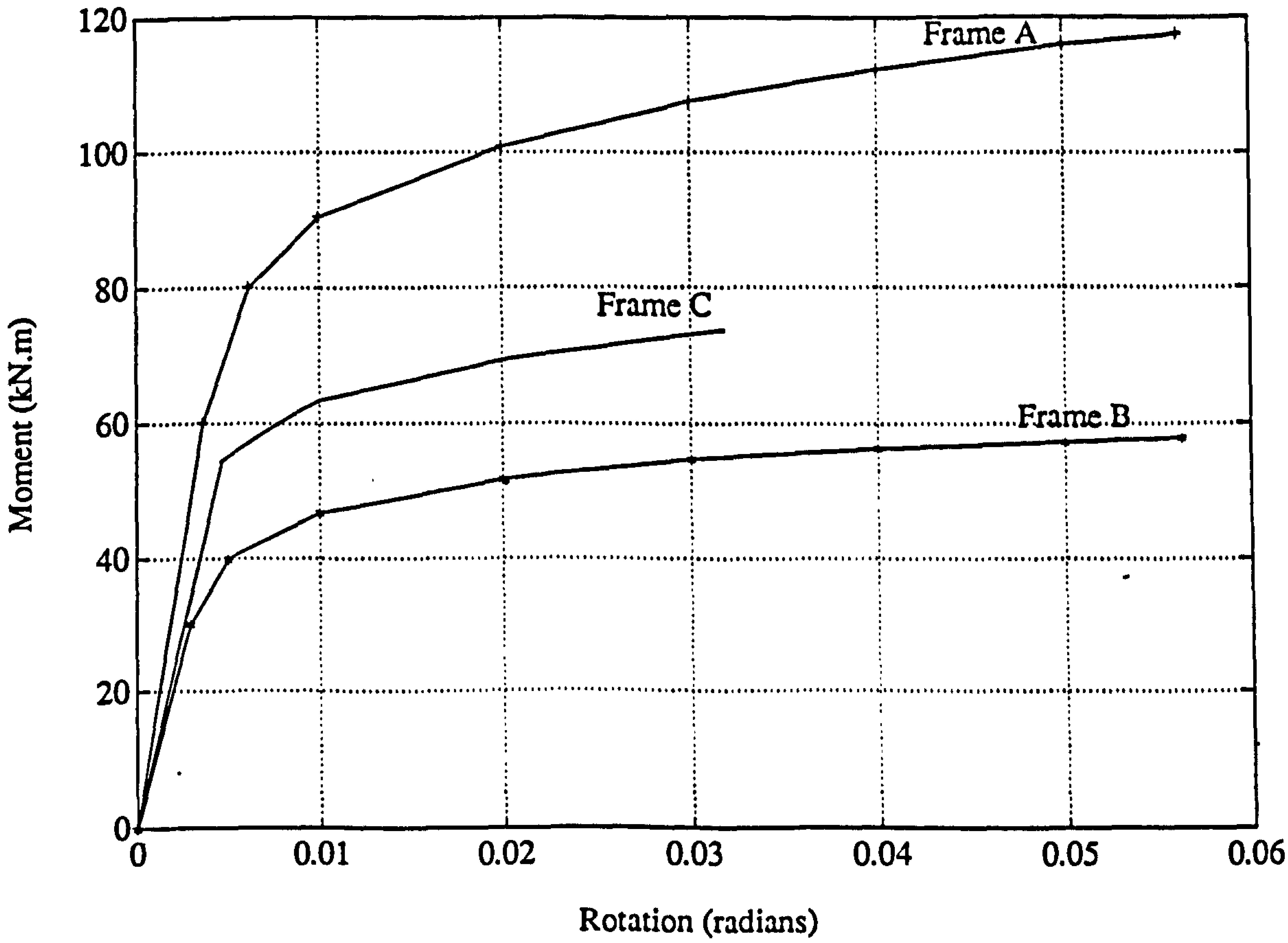
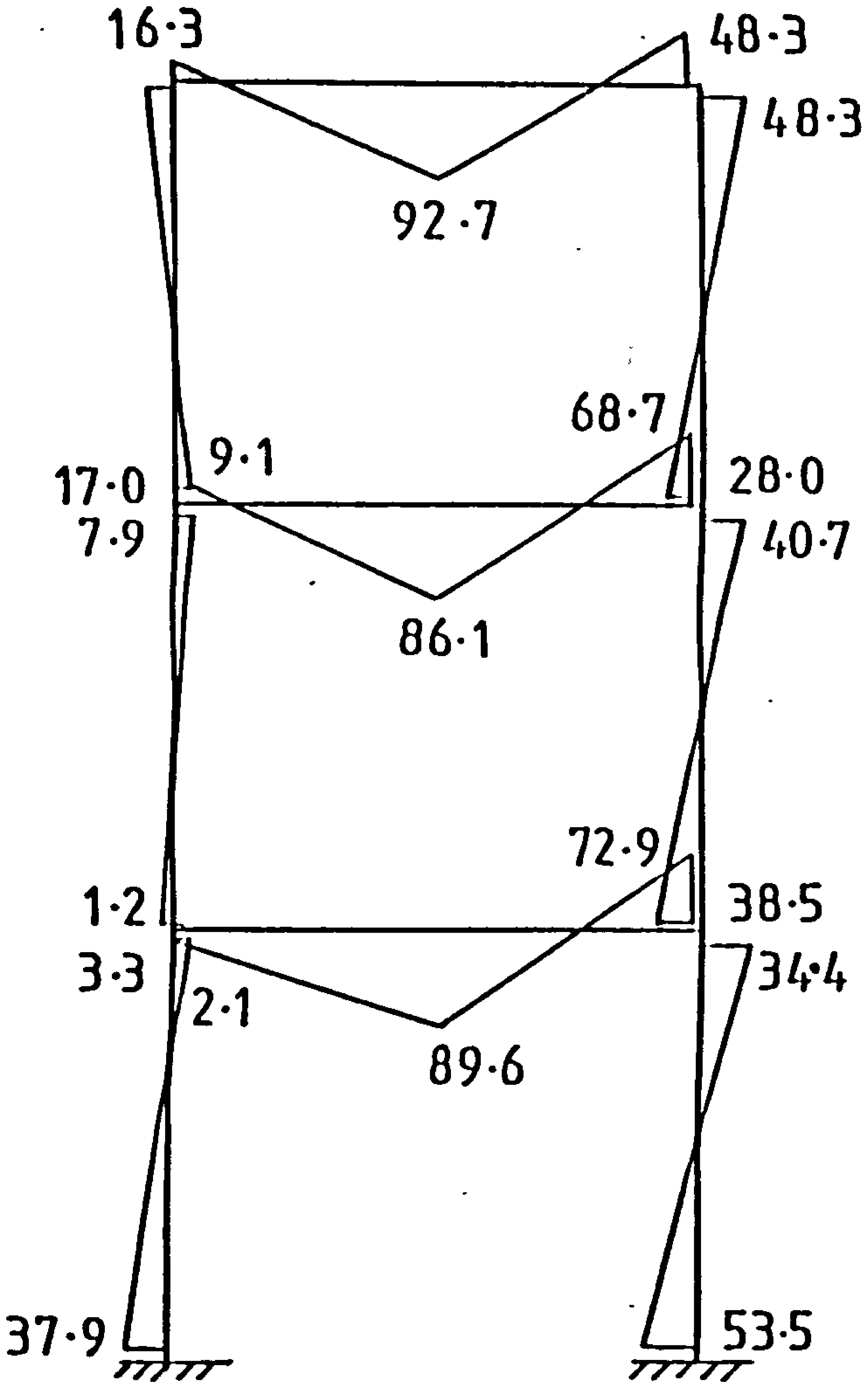
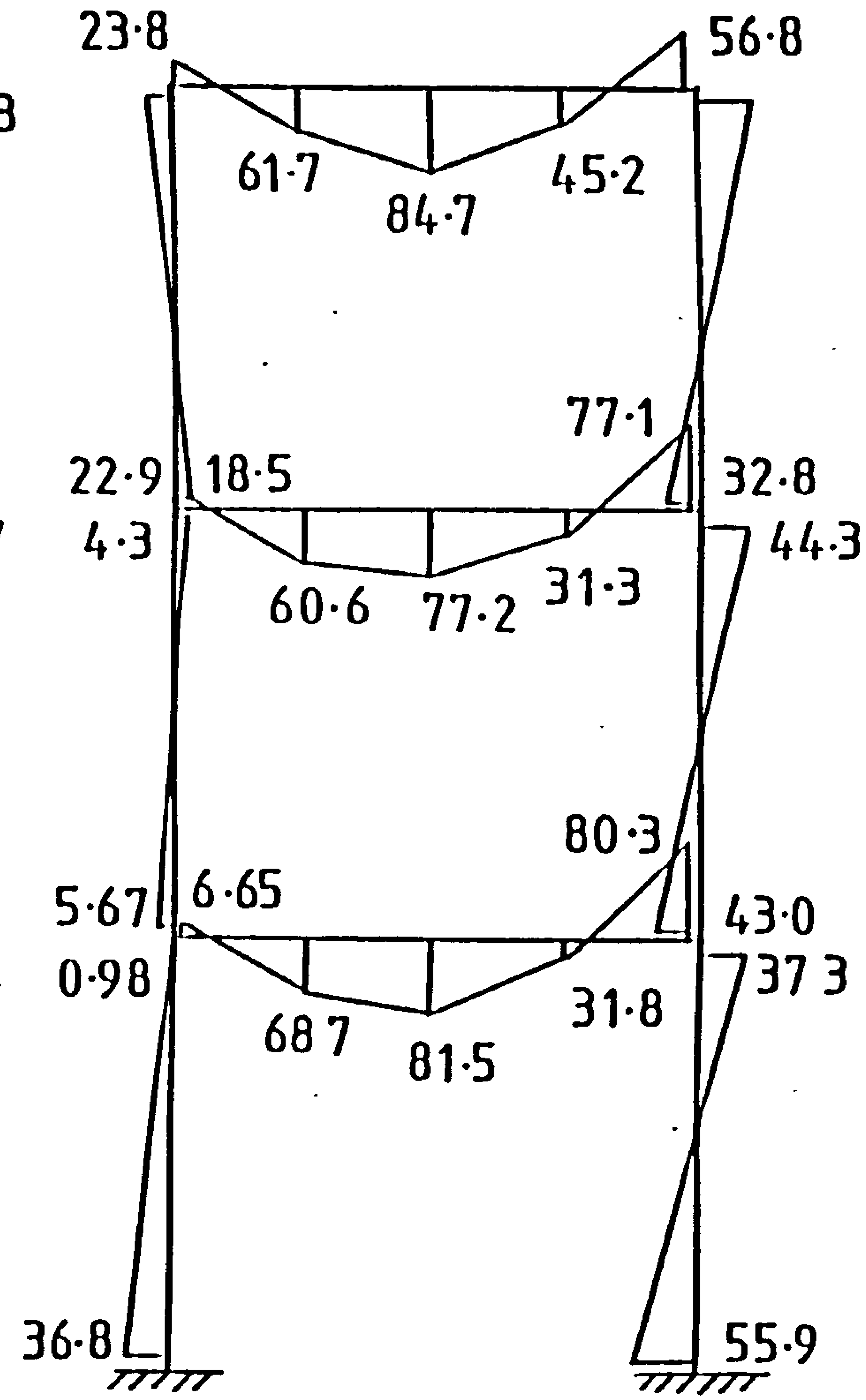


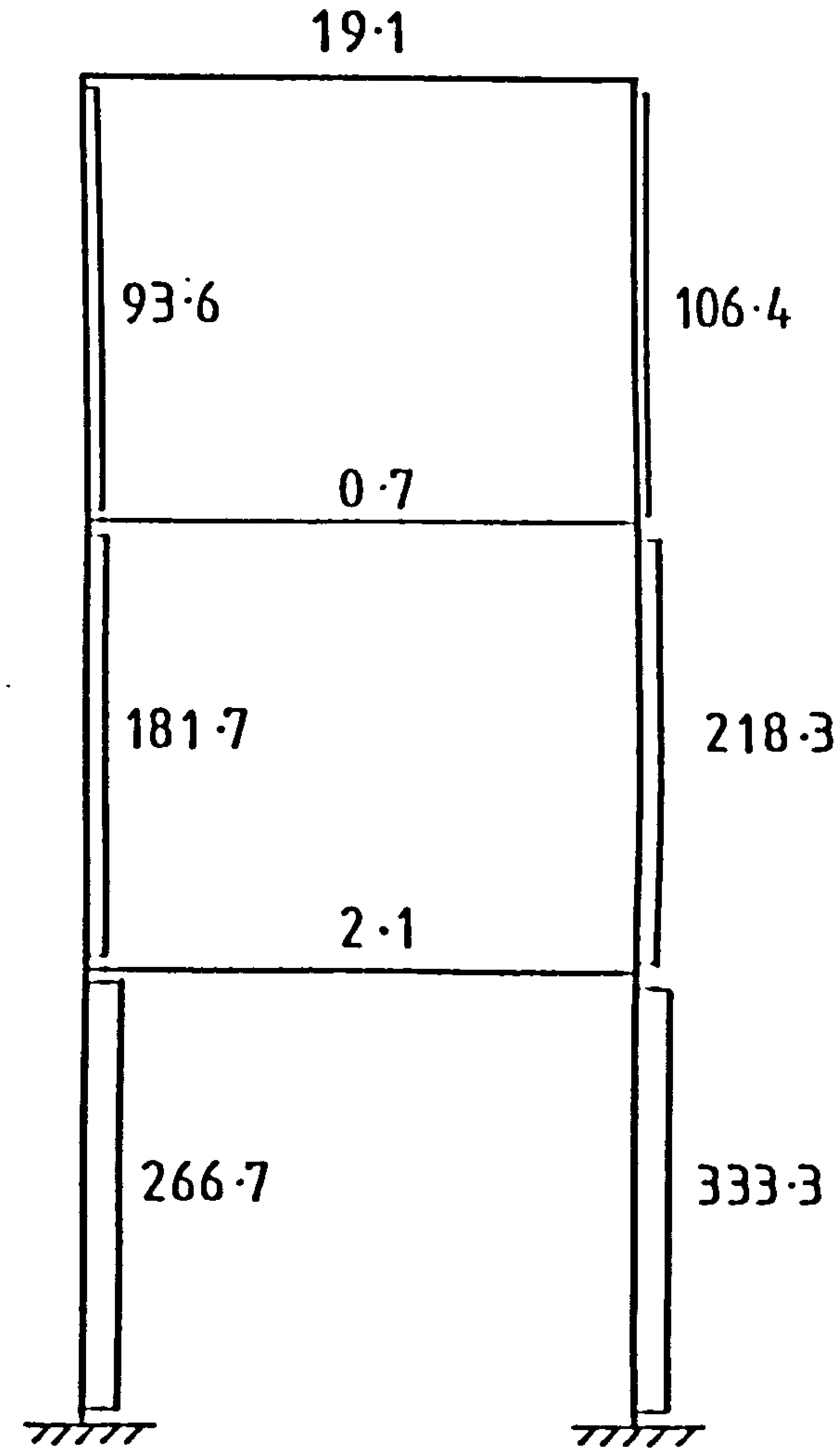
FIG.5-26 Different pattern of loading for Frame A.



Bending moment distribution
(kNm)
Type 1 Loading

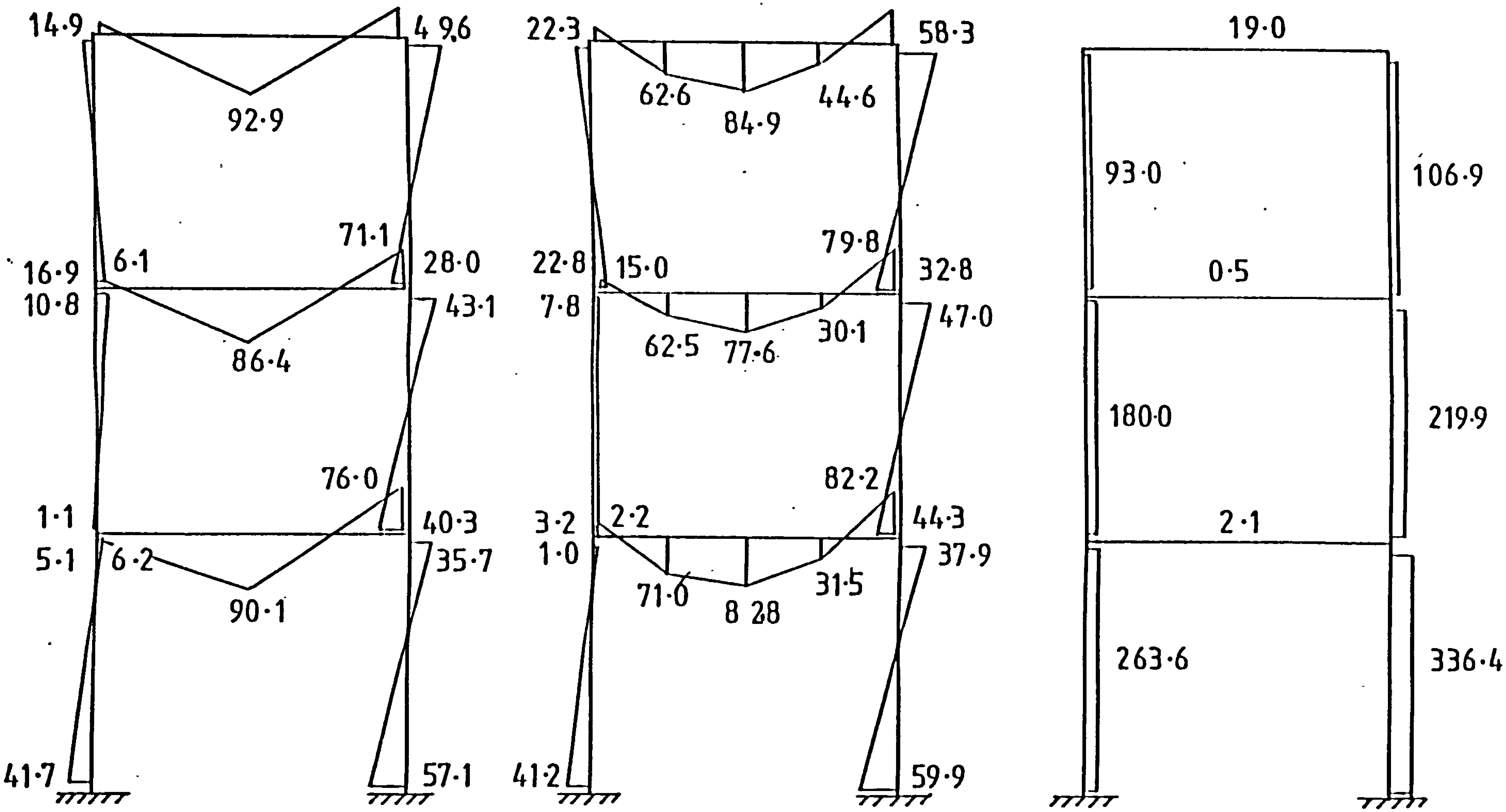


Bending moment distribution
(kNm)
Type 2 Loading



Axial force distribution
(kN)
Type 1 Loading

FIG.5-27 Frame A with semi-rigid joints, 1st order analysis.



Bending moment distribution (kNm)

Type 1 Loading

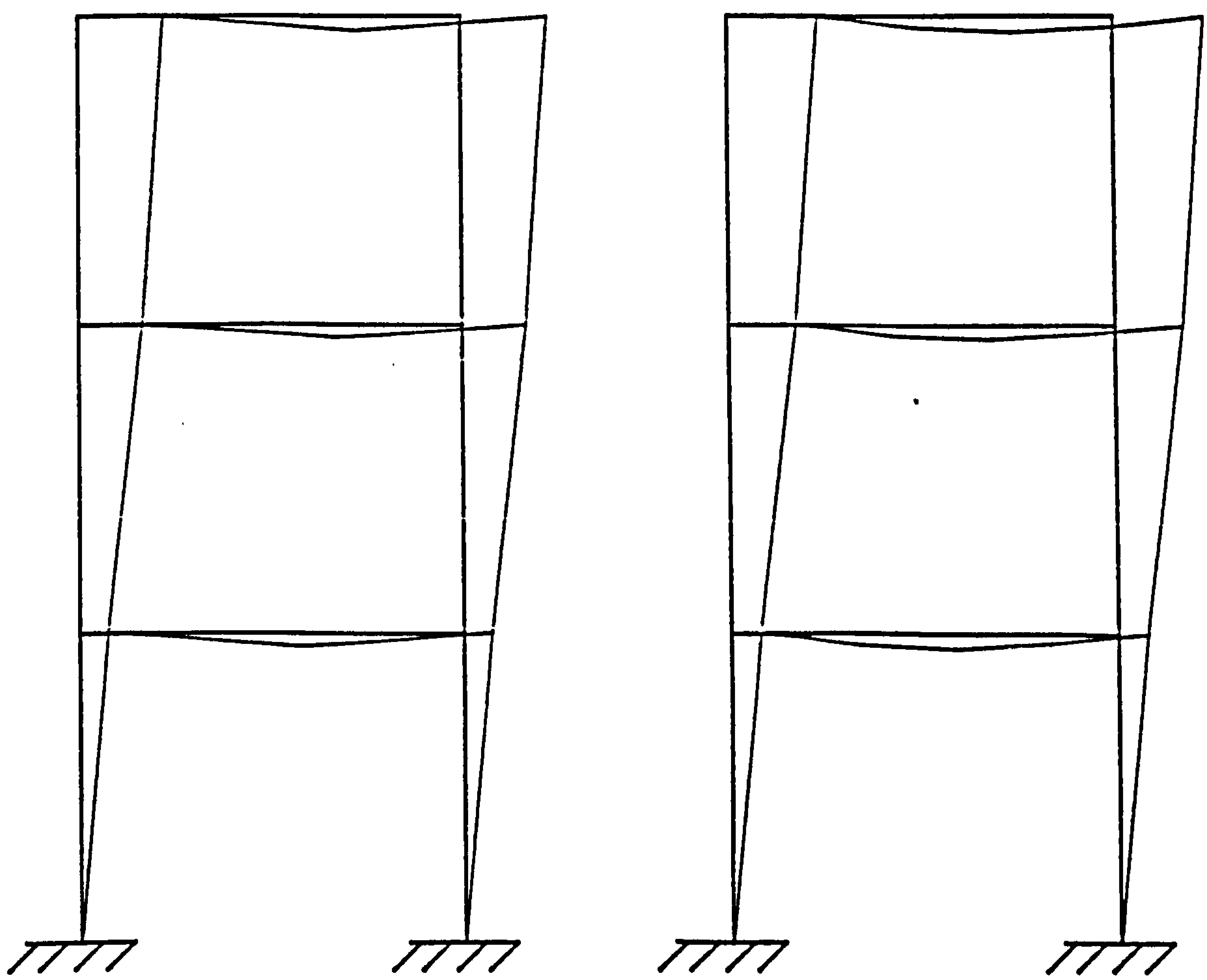
Bending moment distribution (kNm)

Type 2 Loading

Axial force distribution (kNm)

Type 1 Loading

FIG.5-28 Frame A with semi-rigid joints, 2nd order analysis.

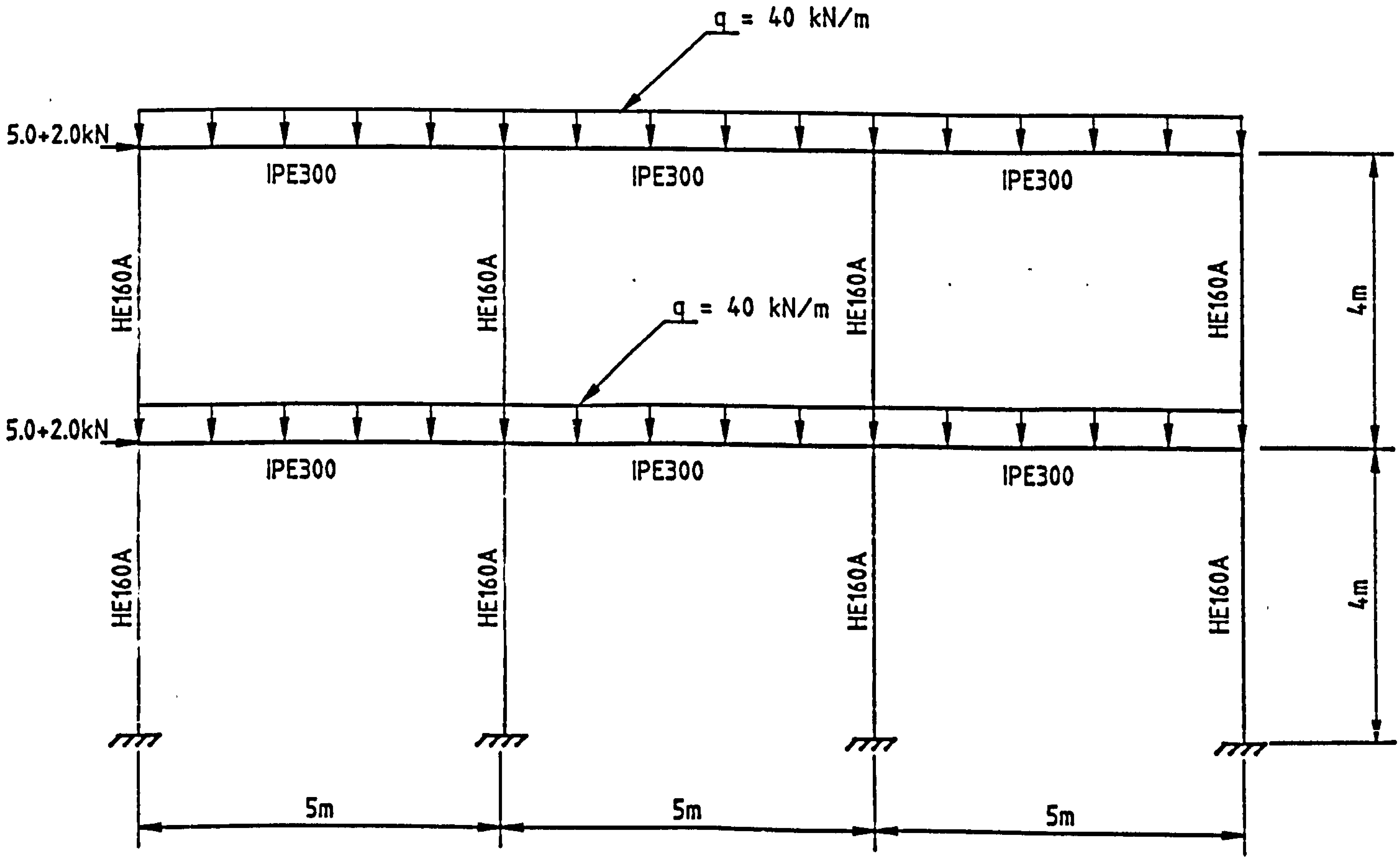


a) Type 1 Loading

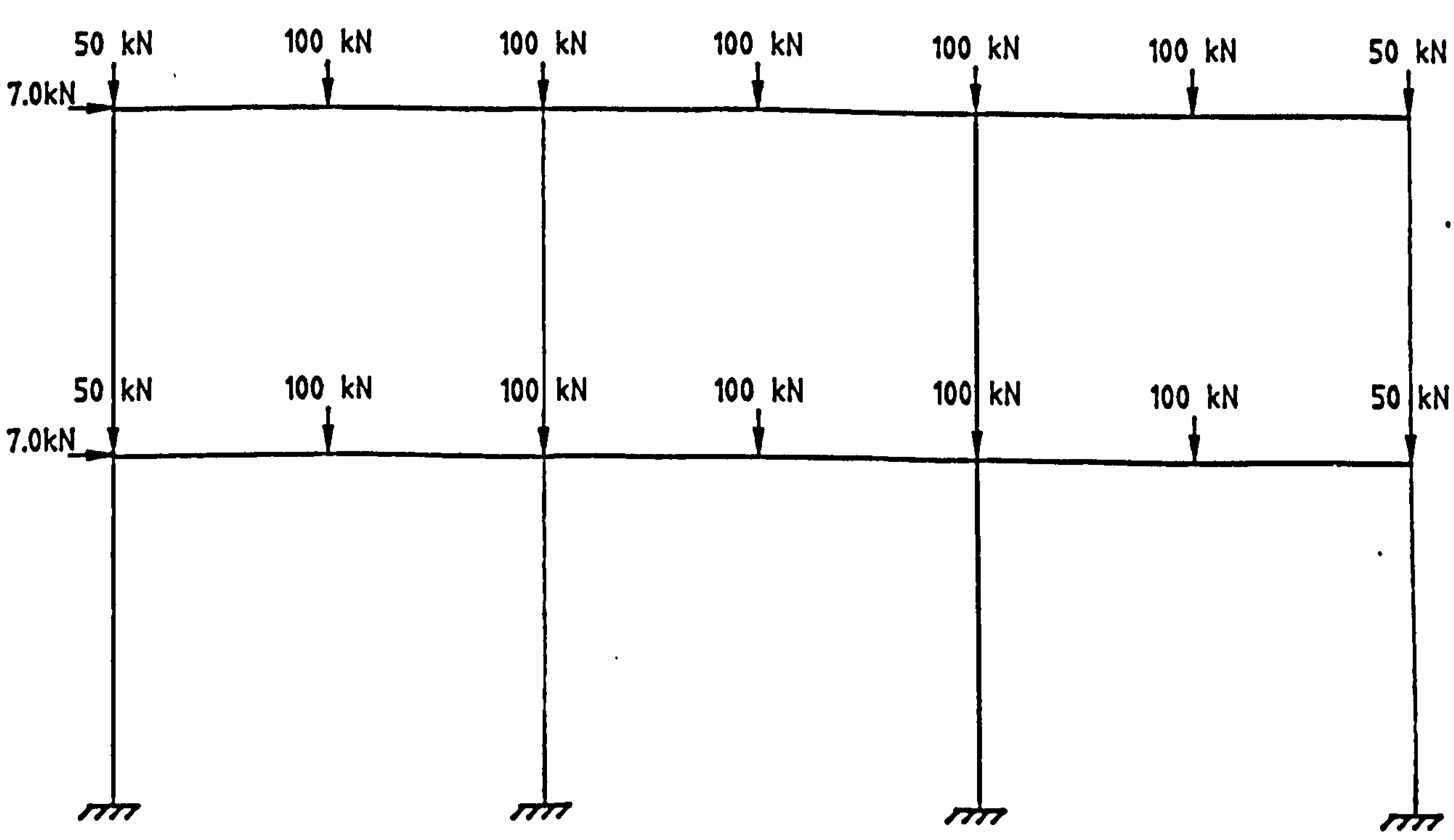
b) Type 2 Loading

Scale :
Frame 1mm to 100mm
Deflected shape 1mm to 5mm

Figure 5.29 Deflected shape of Frame A, 2nd order analysis.

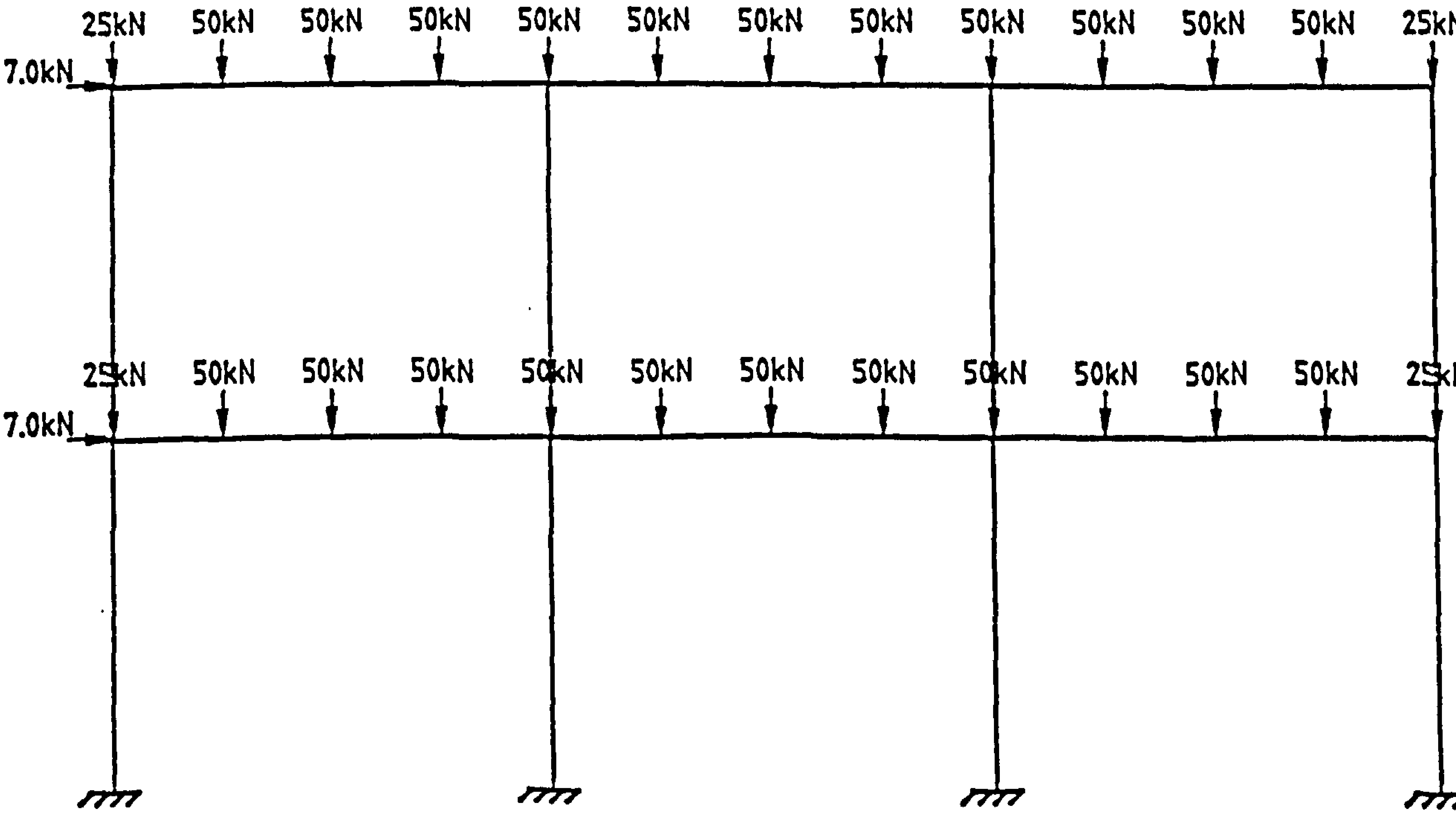


a) Frame configuration



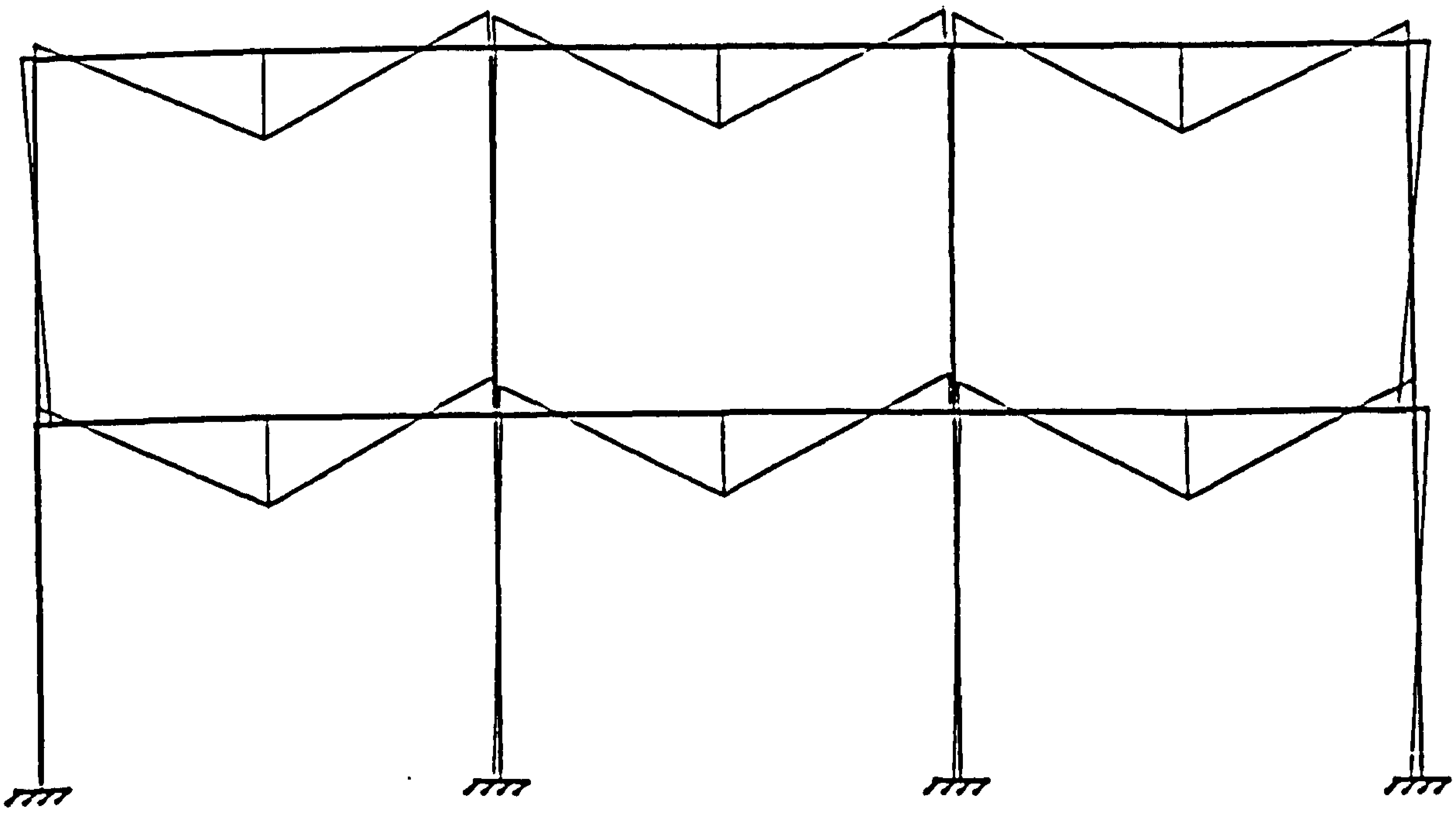
b) Type 1 loading

Figure 5.30 Different pattern of loading for Frame B



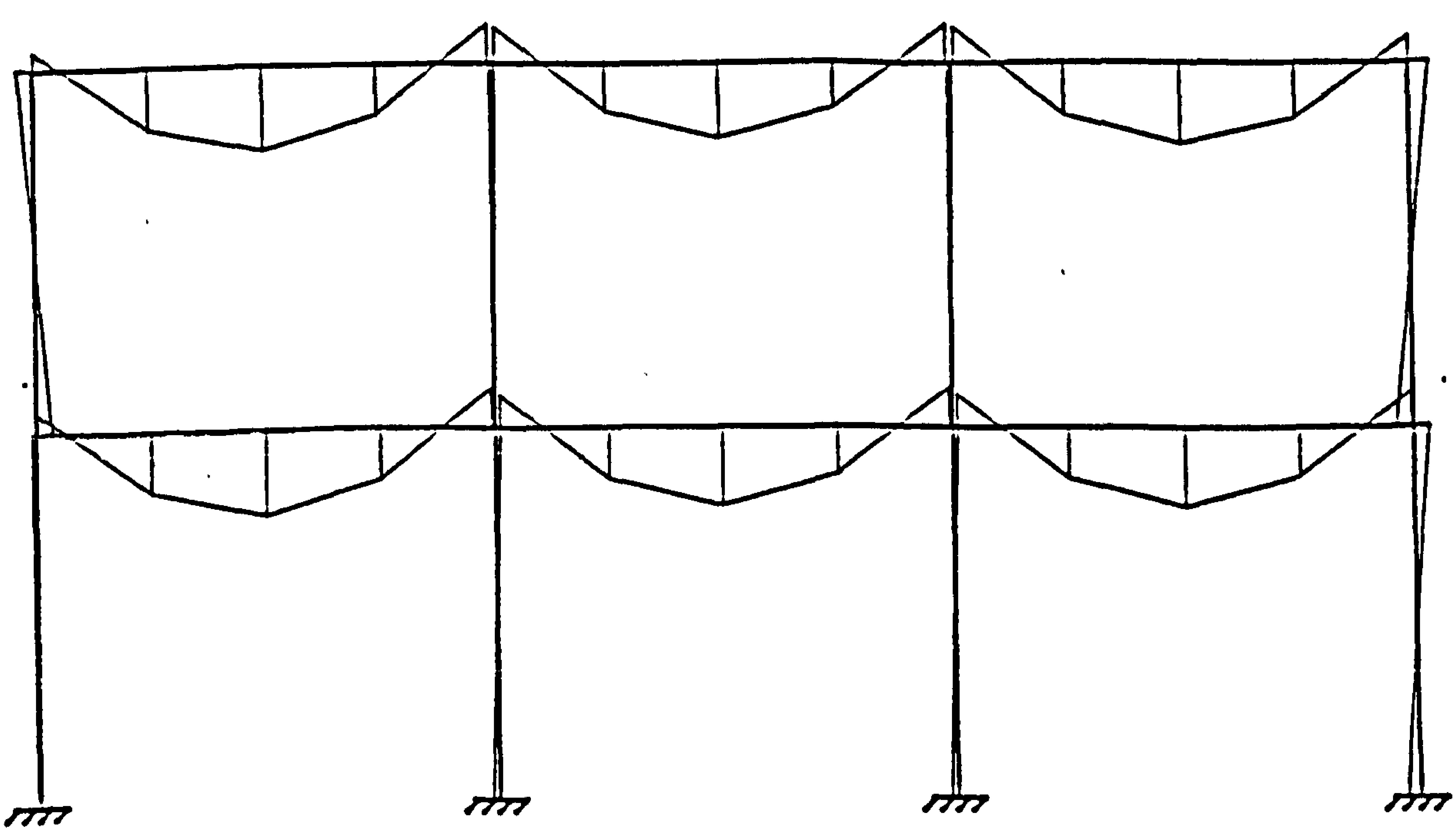
c) Type 2 loading

Figure 5.30 Continued



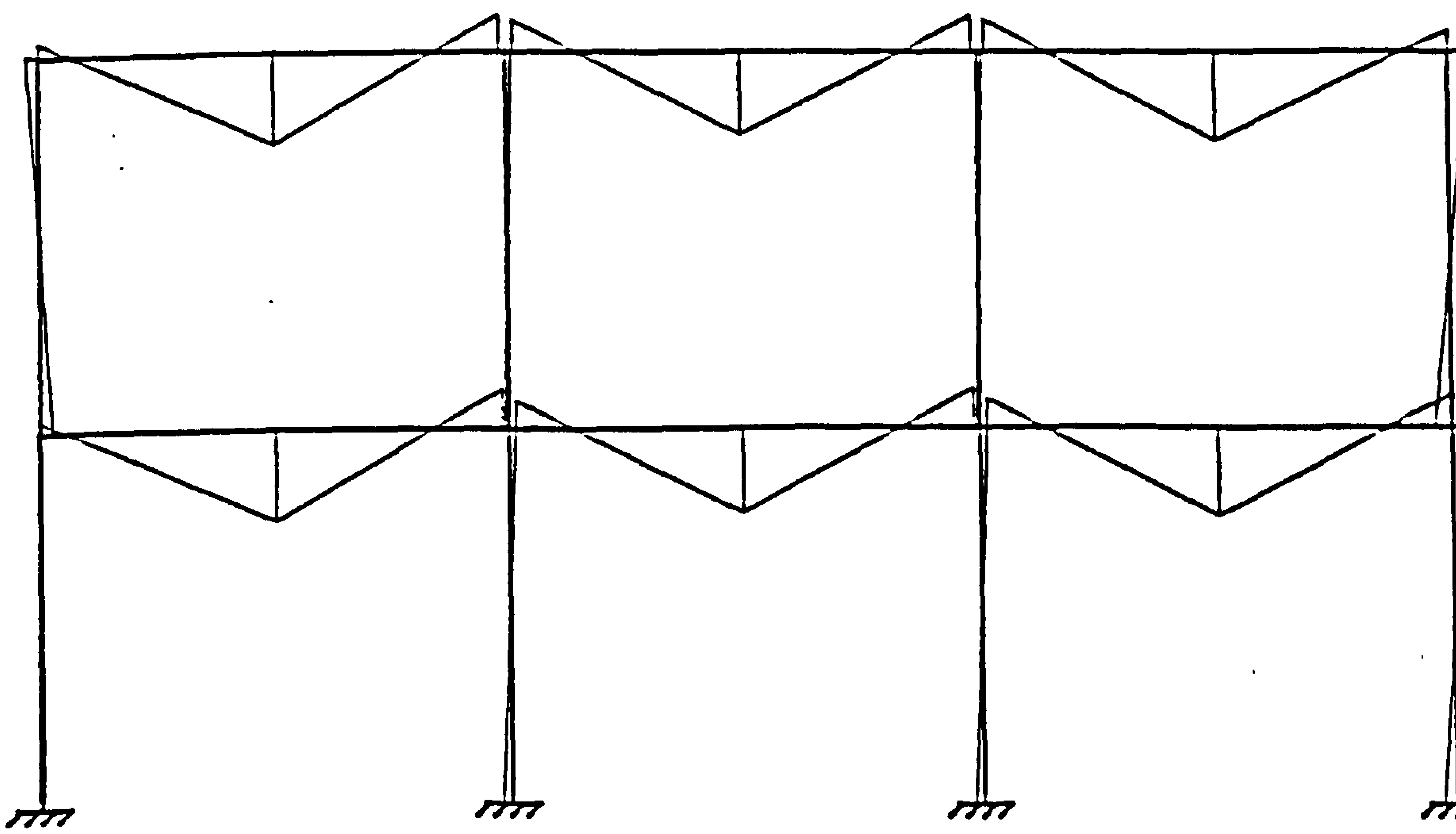
a) Type 1 Loading

Scale :
Frame 1mm to 100mm
Bending moment 1mm to 10kNm



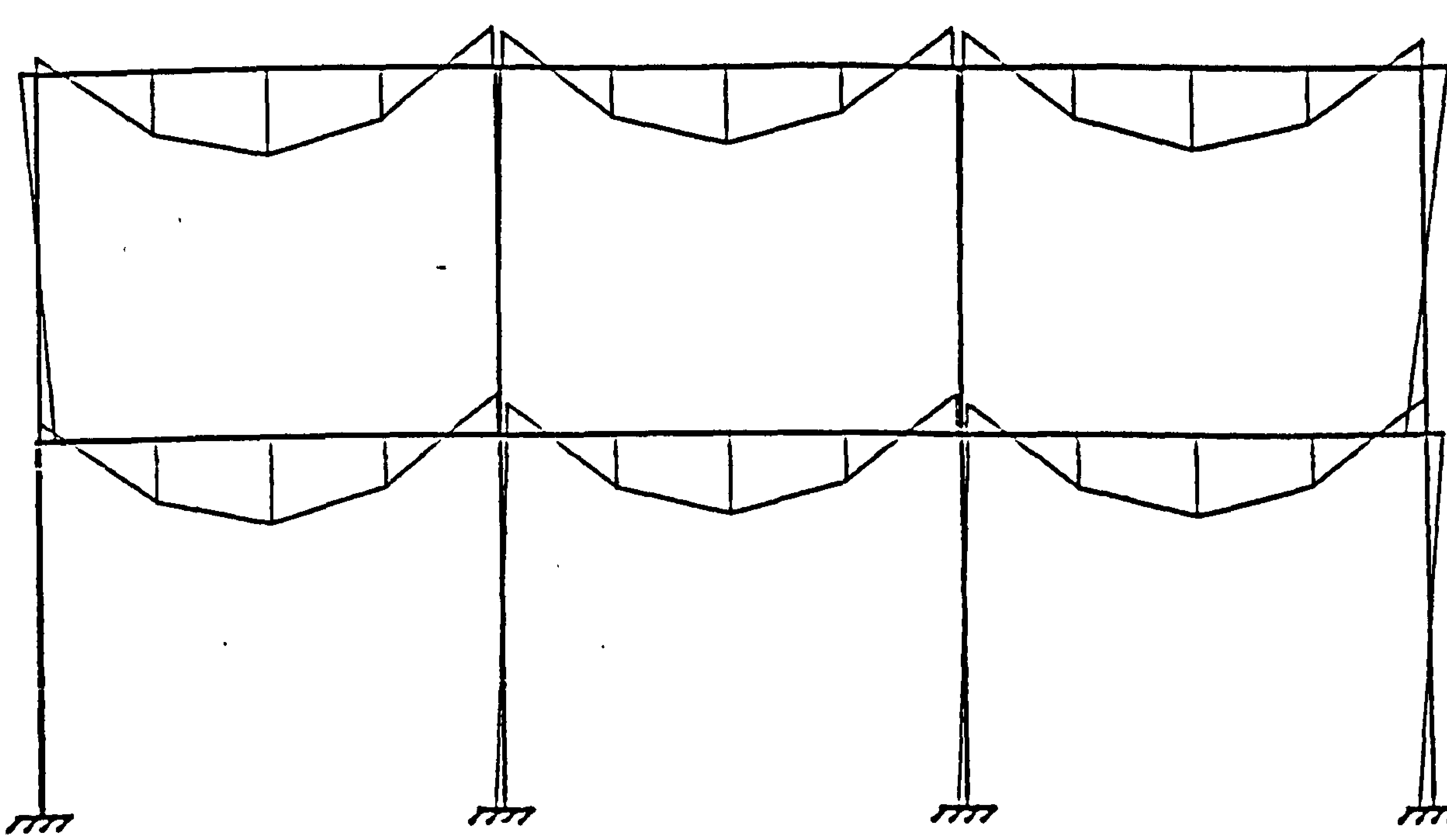
b) Type 2 Loading

Figure 5.31 Bending moment distribution for Frame B with semi-rigid joints, 1st order analysis.



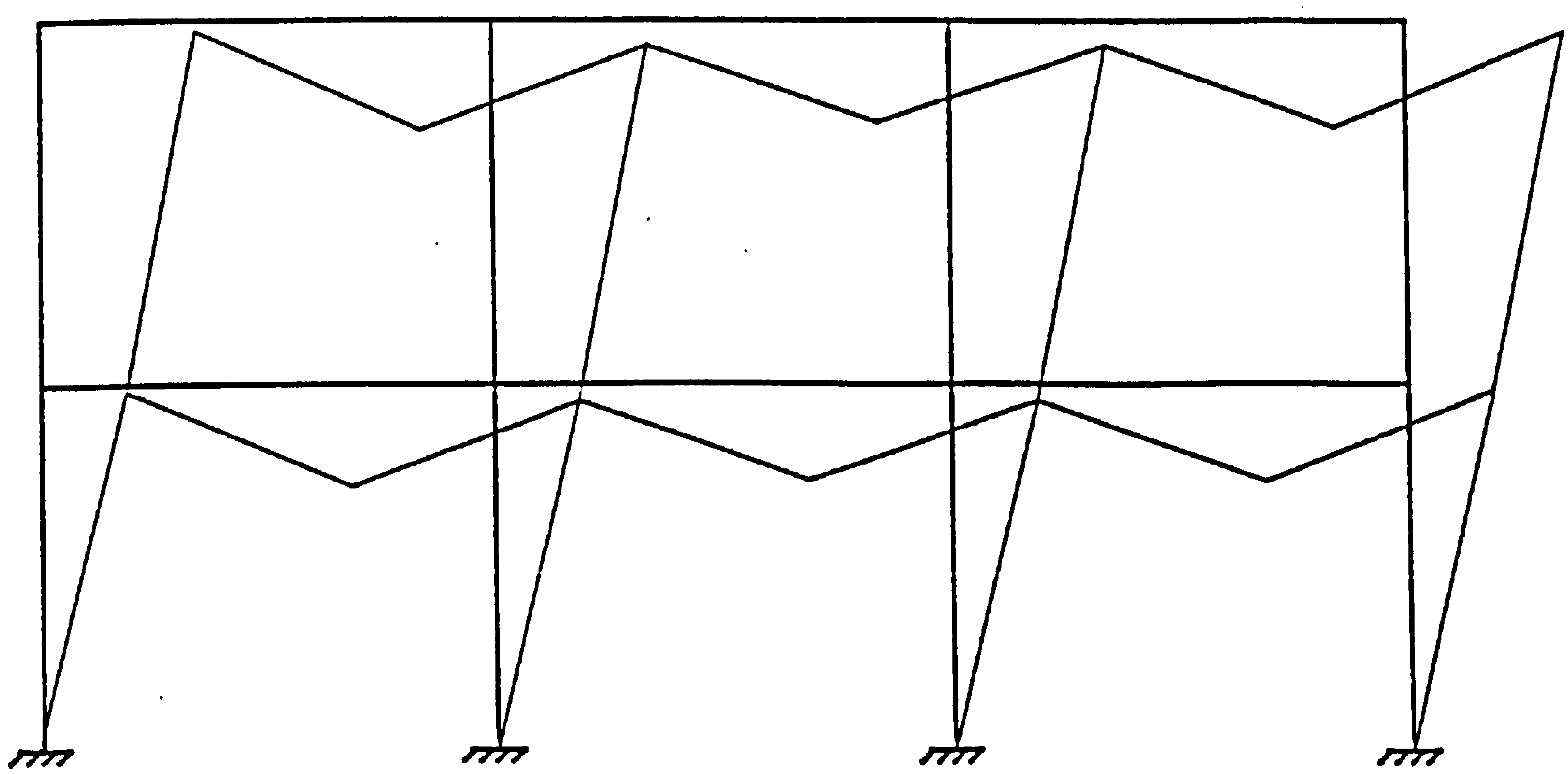
a) Type 1 Loading

Scale :
Frame 1mm to 100mm
Bending moment 1mm to 10kNm



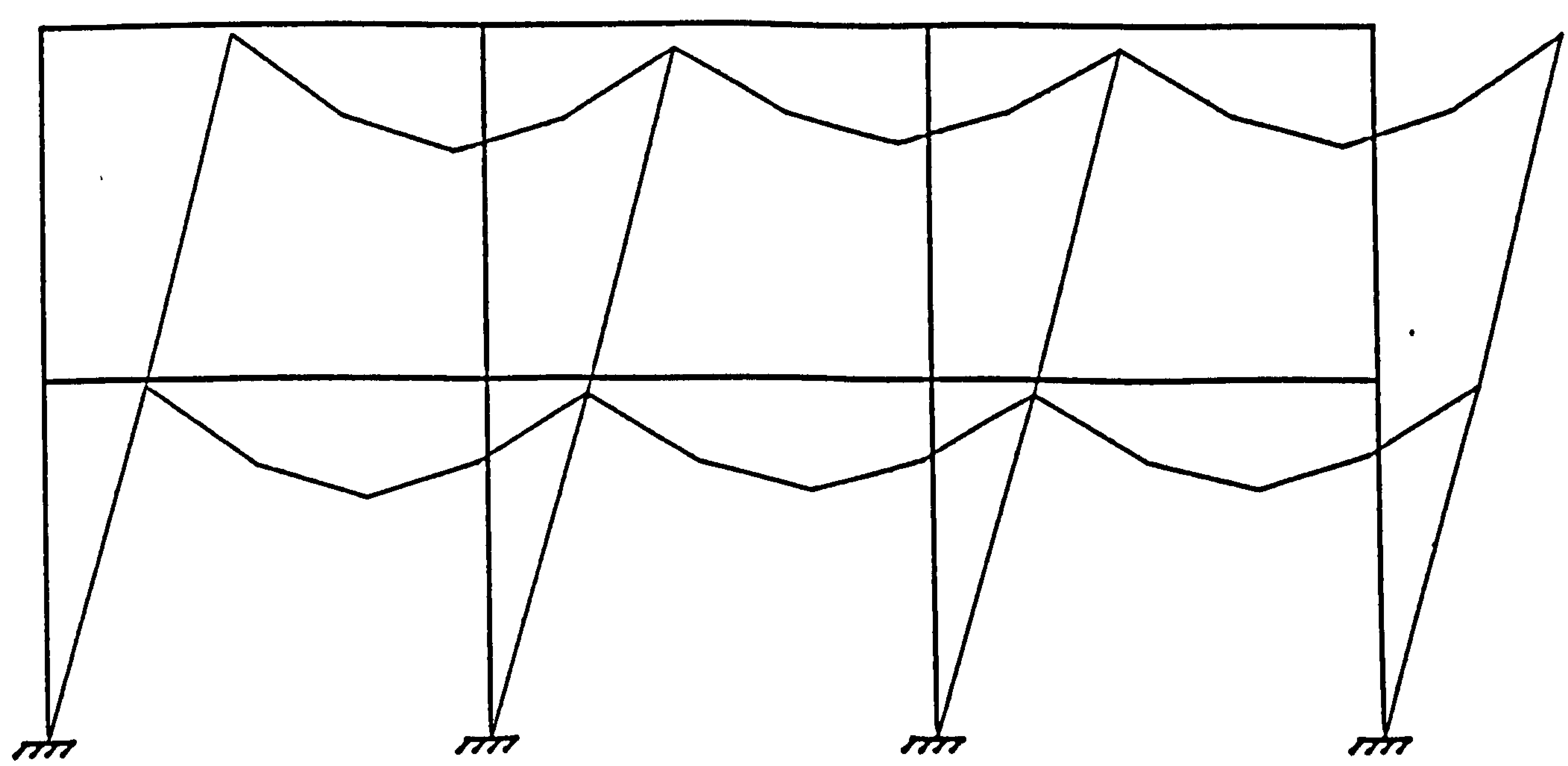
b) Type 2 Loading

Figure 5.32 Bending moment distribution for Frame B
with semi-rigid joints, 2nd order analysis



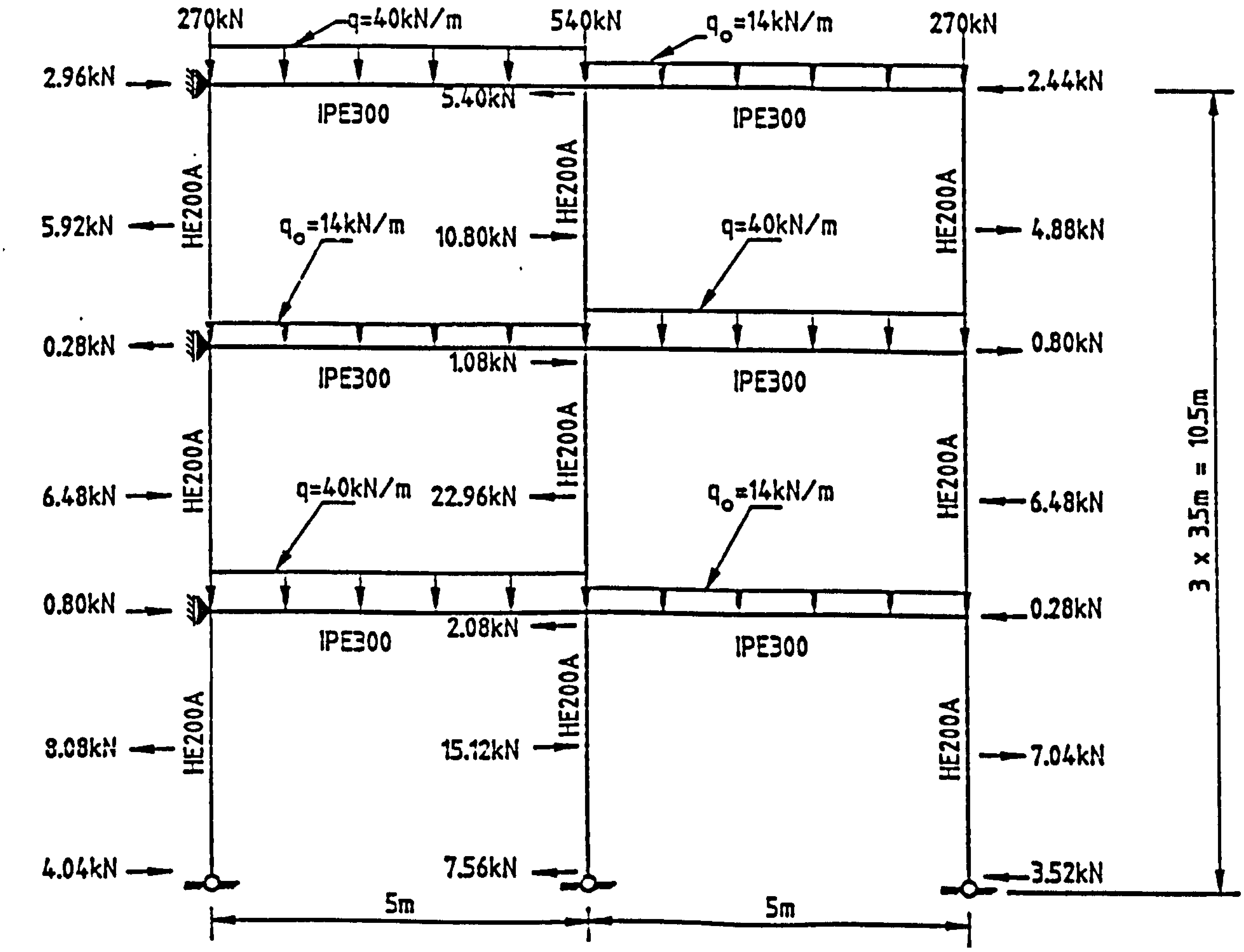
a) Type 1 Loading

Scale :
Frame 1mm to 100mm
Deflected shape 1mm to 1mm

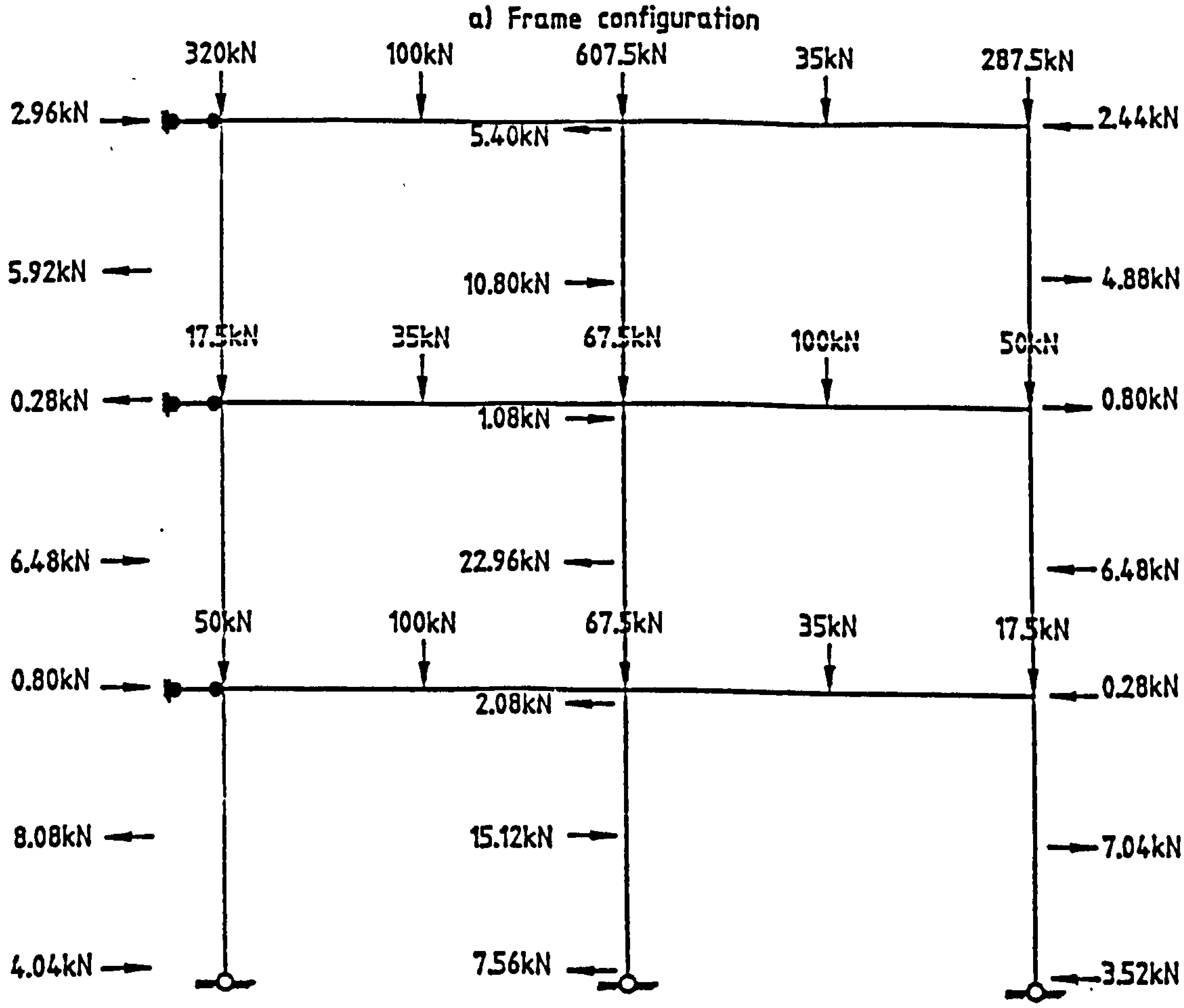


b) Type 2 Loading

Figure 5.33 Deflected shape of Frame B, 2nd order analysis.

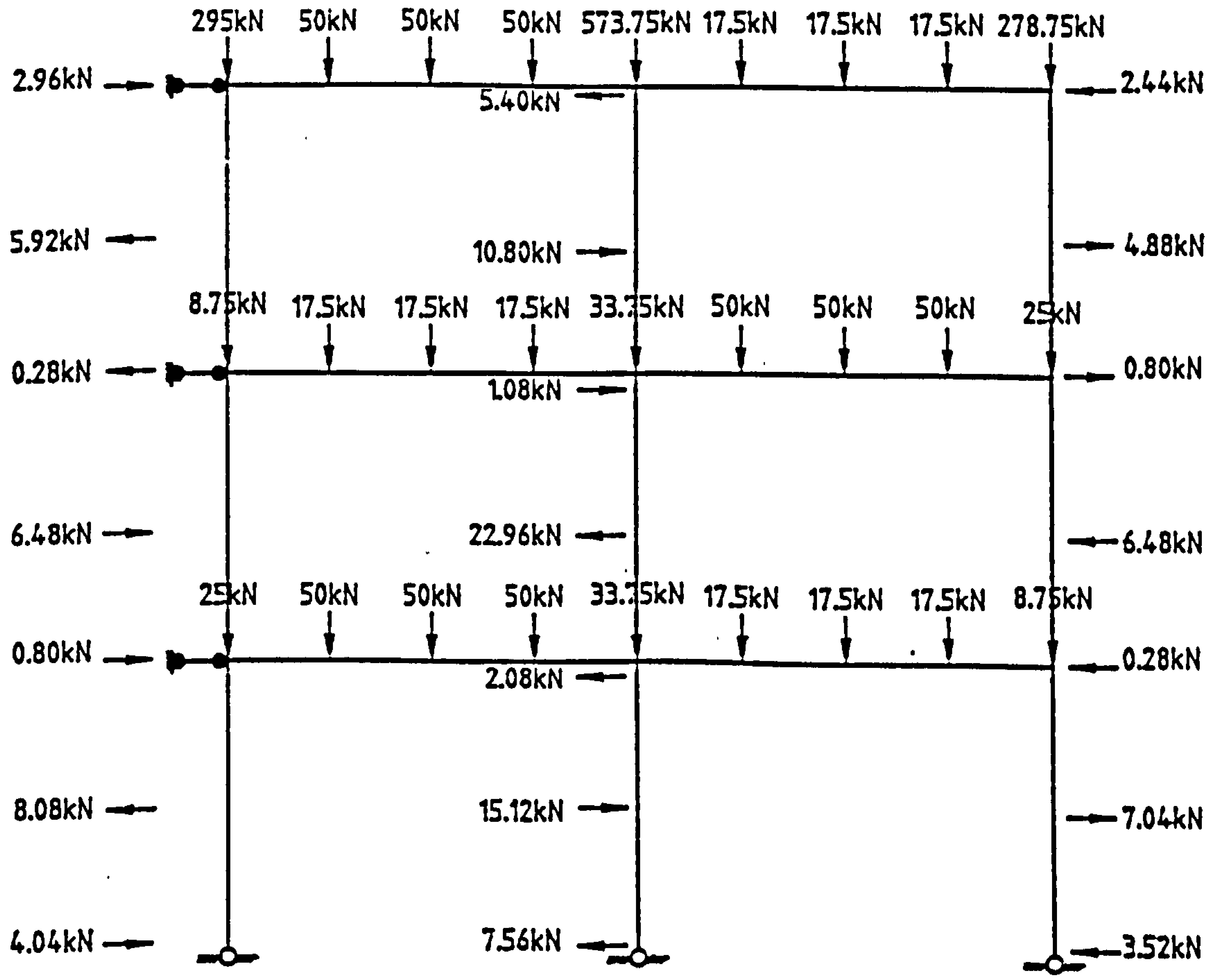


a) Frame configuration



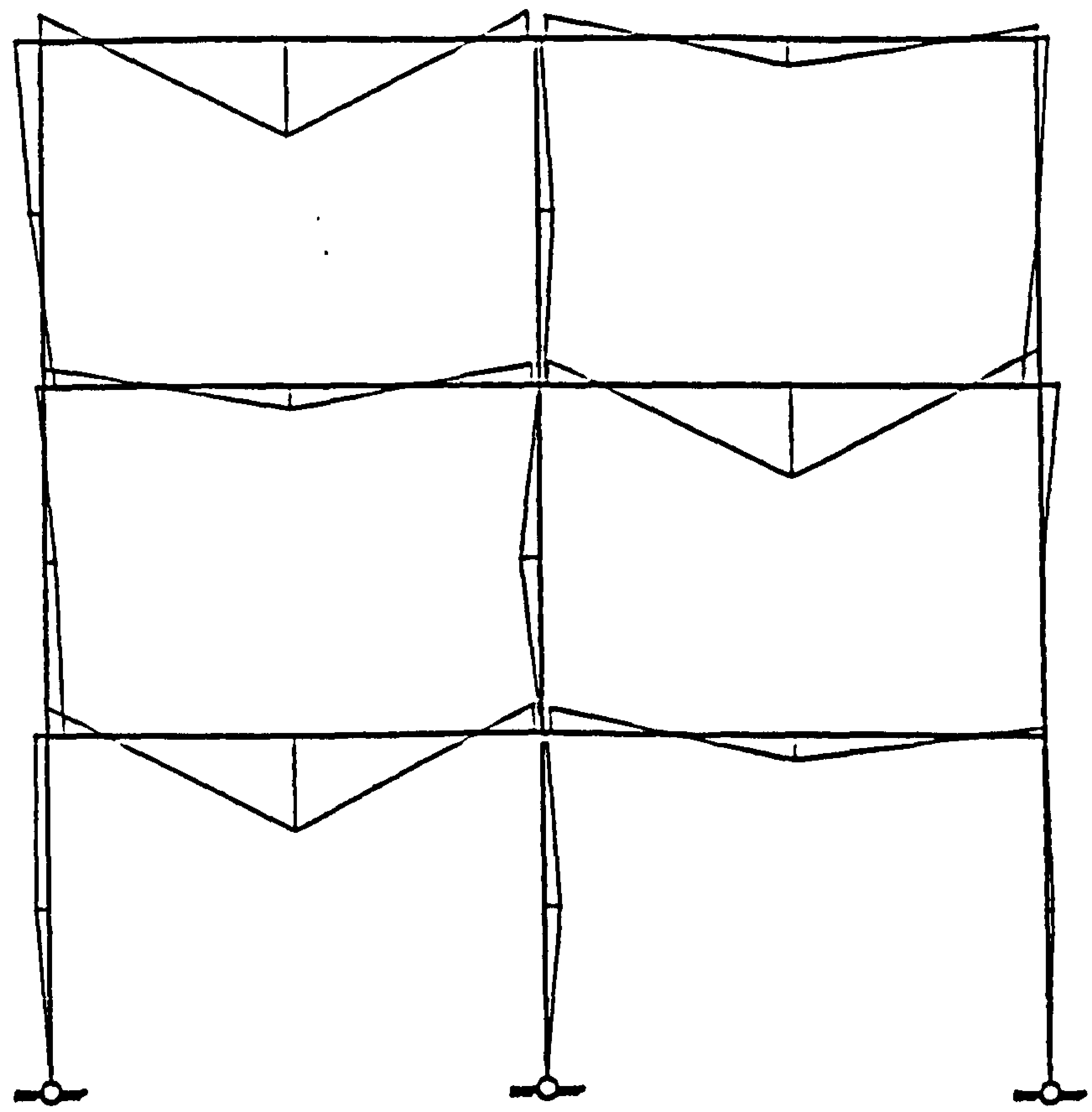
b) Type 1 Loading

Figure 5.34 Different pattern of loading for Frame C



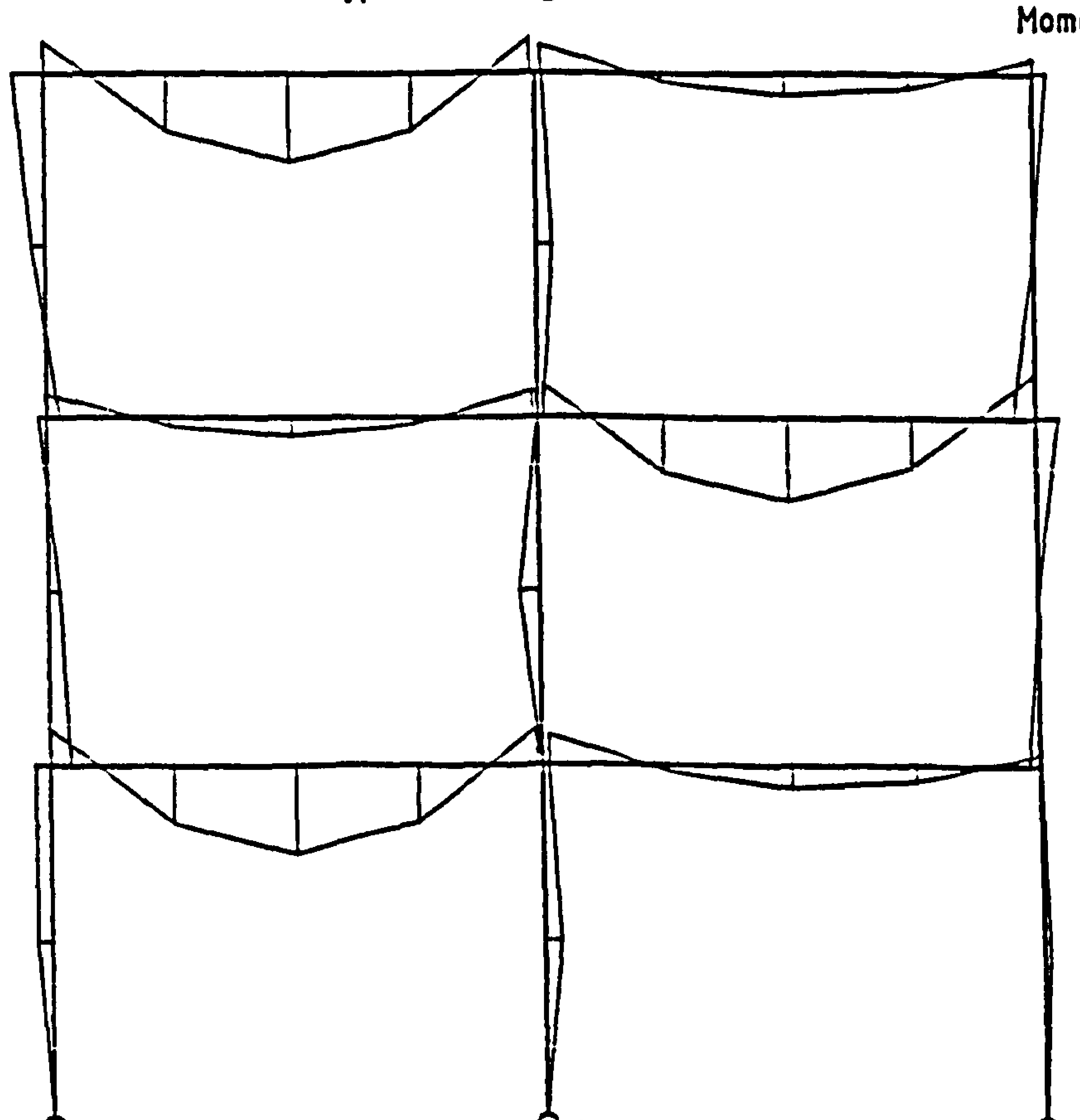
c) Type 2 Loading

Figure 5.34 Continued



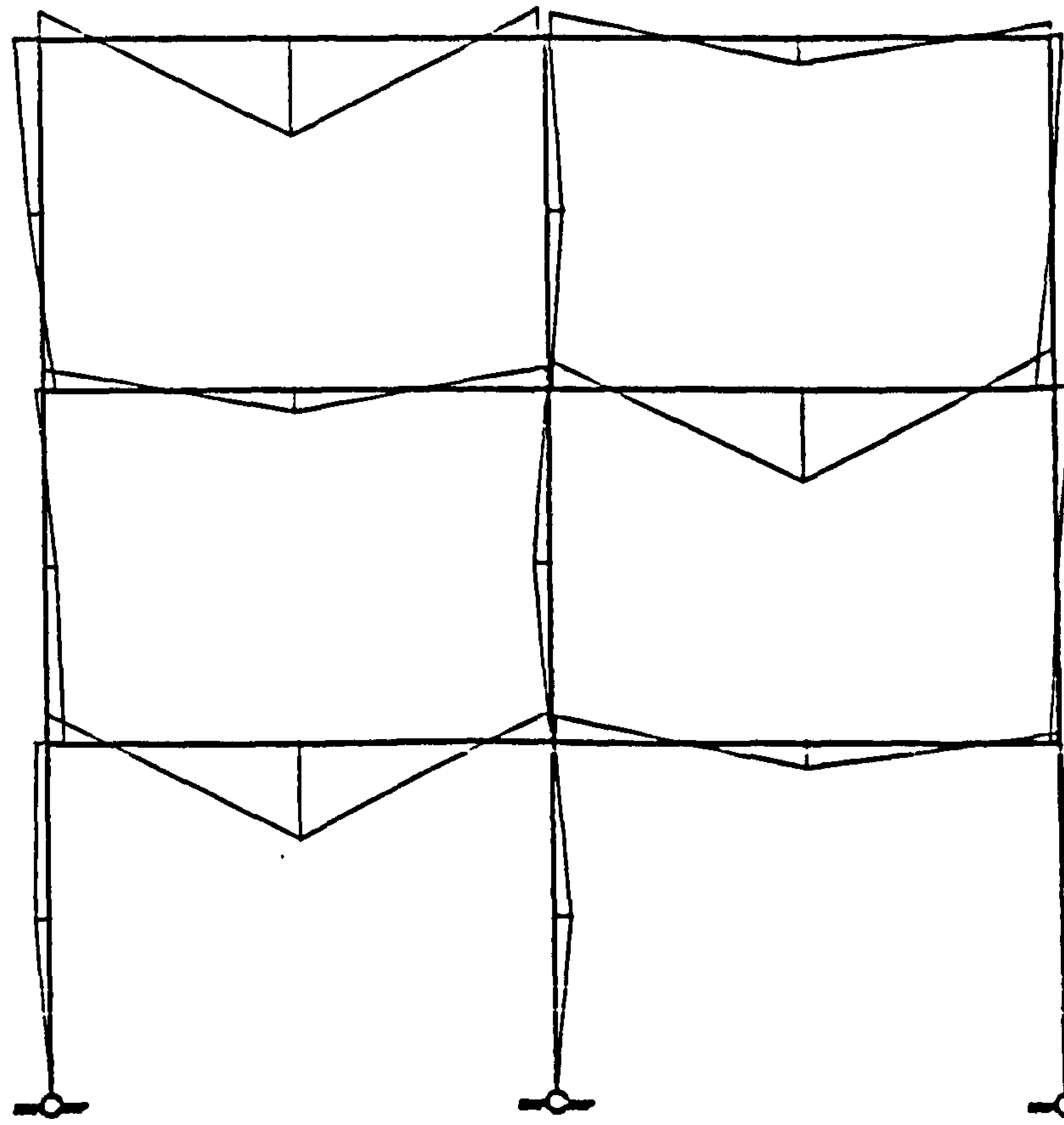
a) Type 1 Loading

Scale :
Frame 1mm to 100mm
Moment 1mm to 10kNm



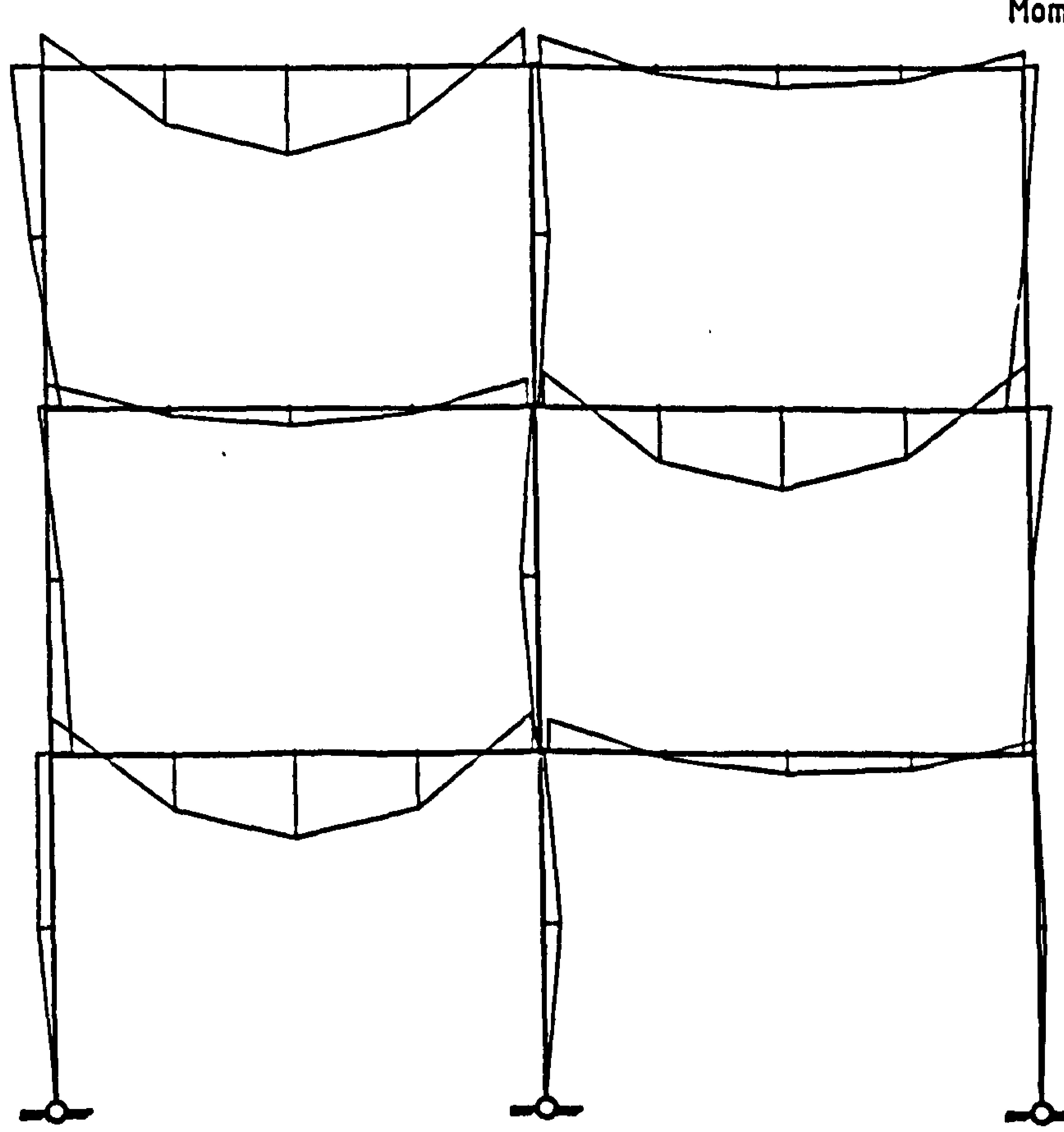
b) Type 2 Loading

Figure 5.35 Bending moment distribution for Frame C with semi-rigid joints, 1st order analysis.



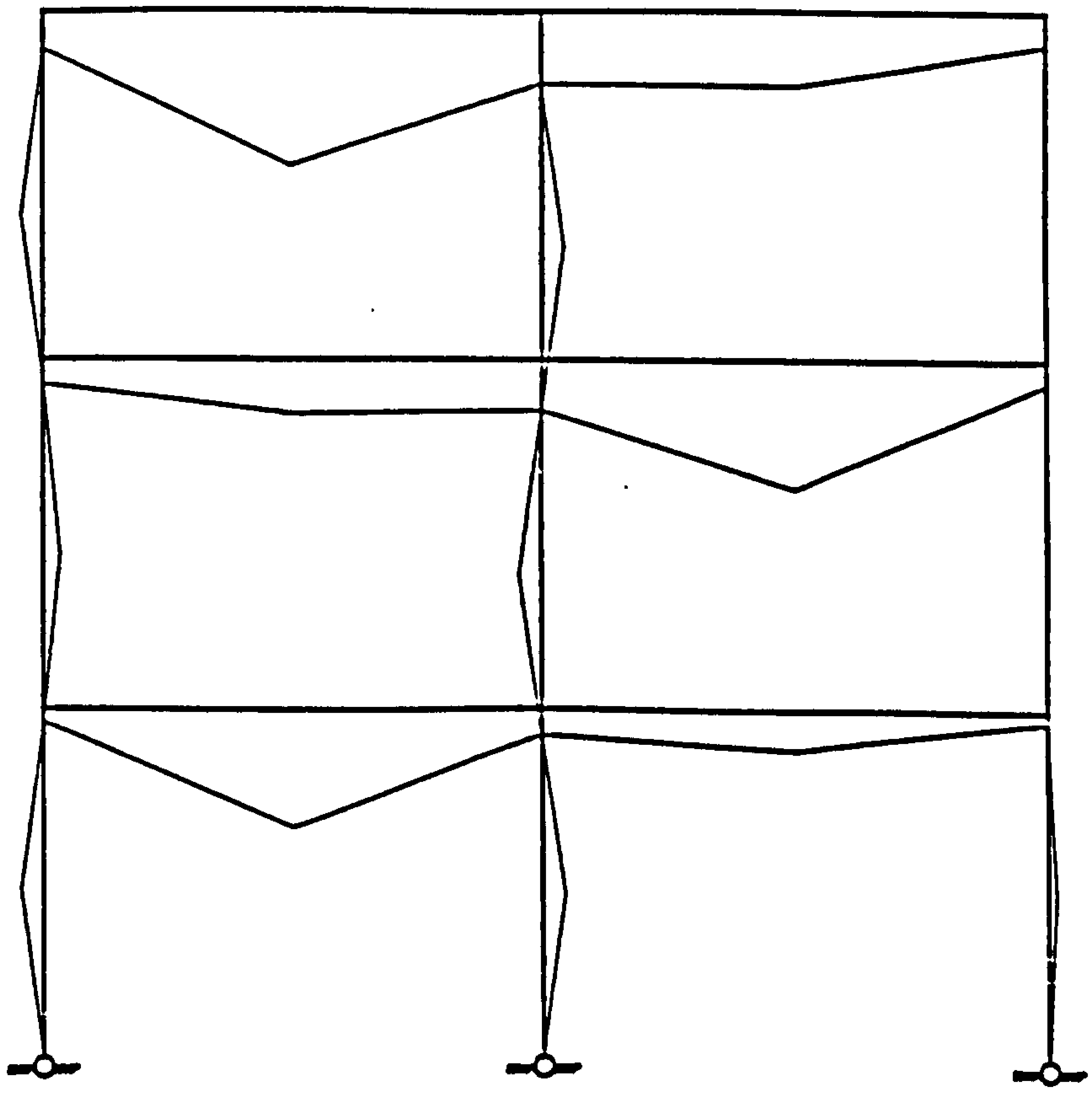
a) Type 1 Loading

Scale :
Frame 1mm to 100mm
Moment 1mm to 10kNm



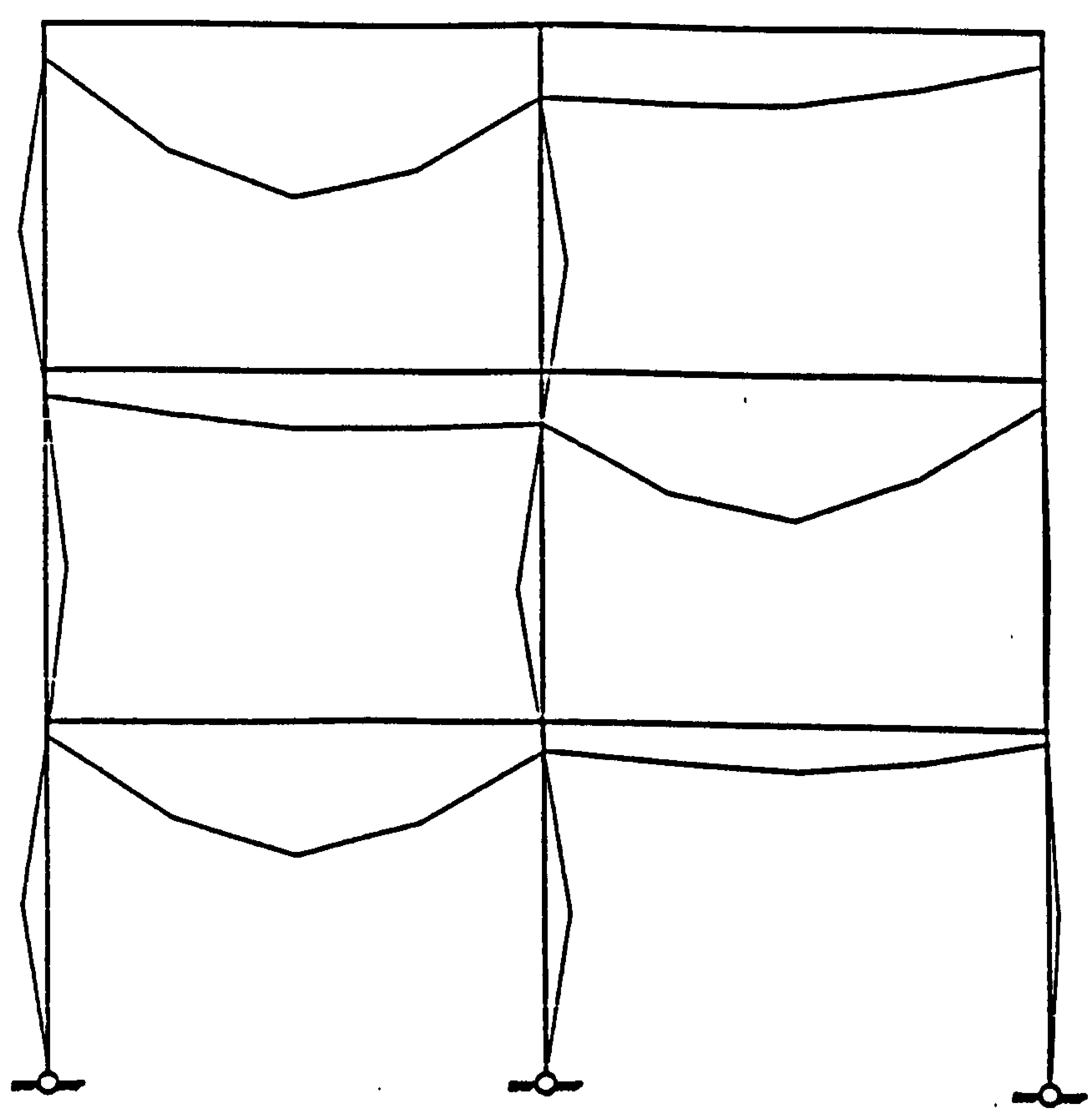
b) Type 2 Loading

Figure 5.36 Bending moment distribution for Frame C with semi-rigid joints, 2nd order analysis.



a) Type 1 Loading

Scale :
Frame 1mm to 100mm
Deflected shape 1mm to 1mm



b) Type 2 Loading

Figure 5.37 Deflected shape of Frame C, 2nd order analysis.

COMPARISON OF DIFFERENT METHODS OF ANALYSIS

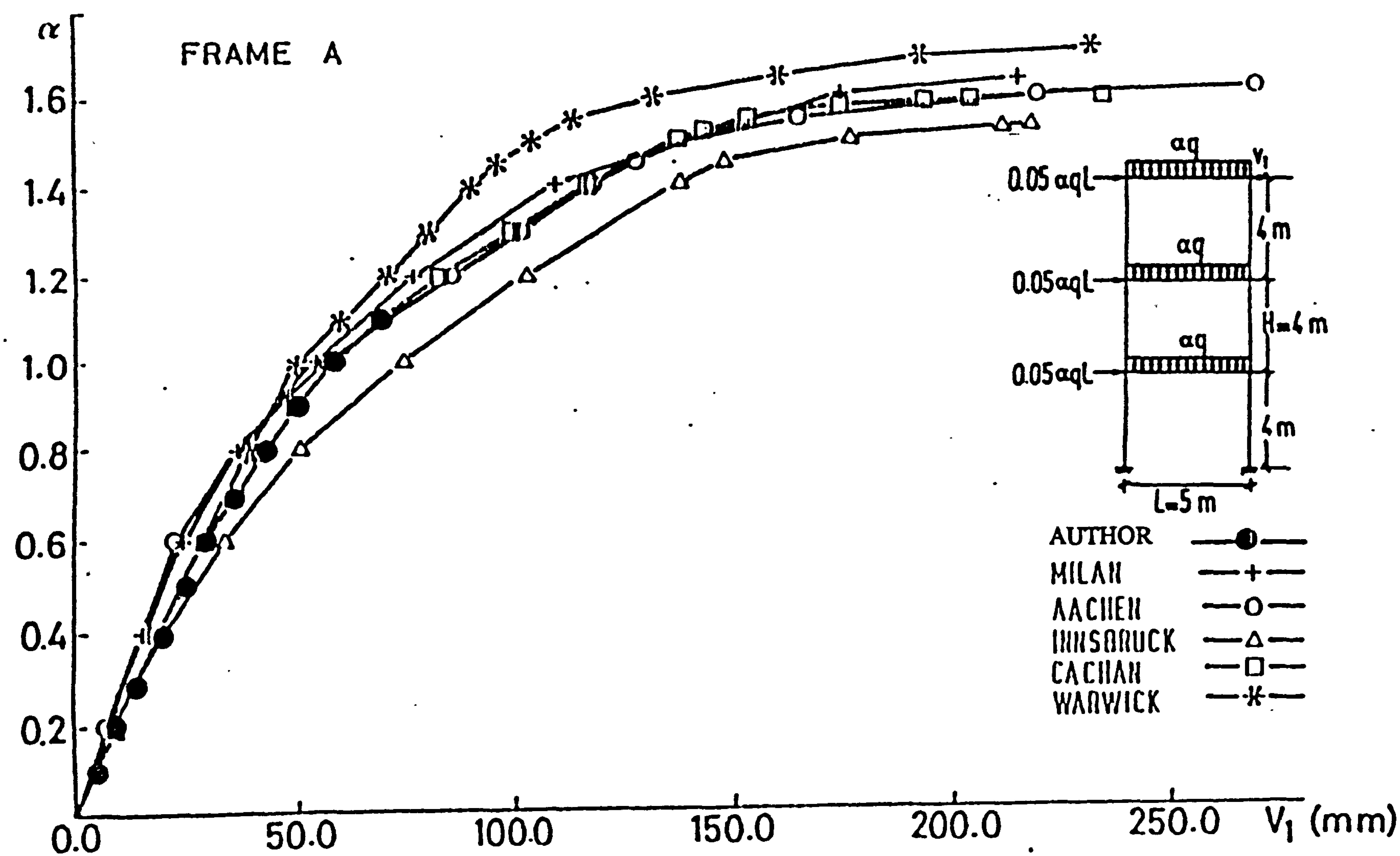


FIG.5-38 Comparison of different methods of analysis for Frame A.

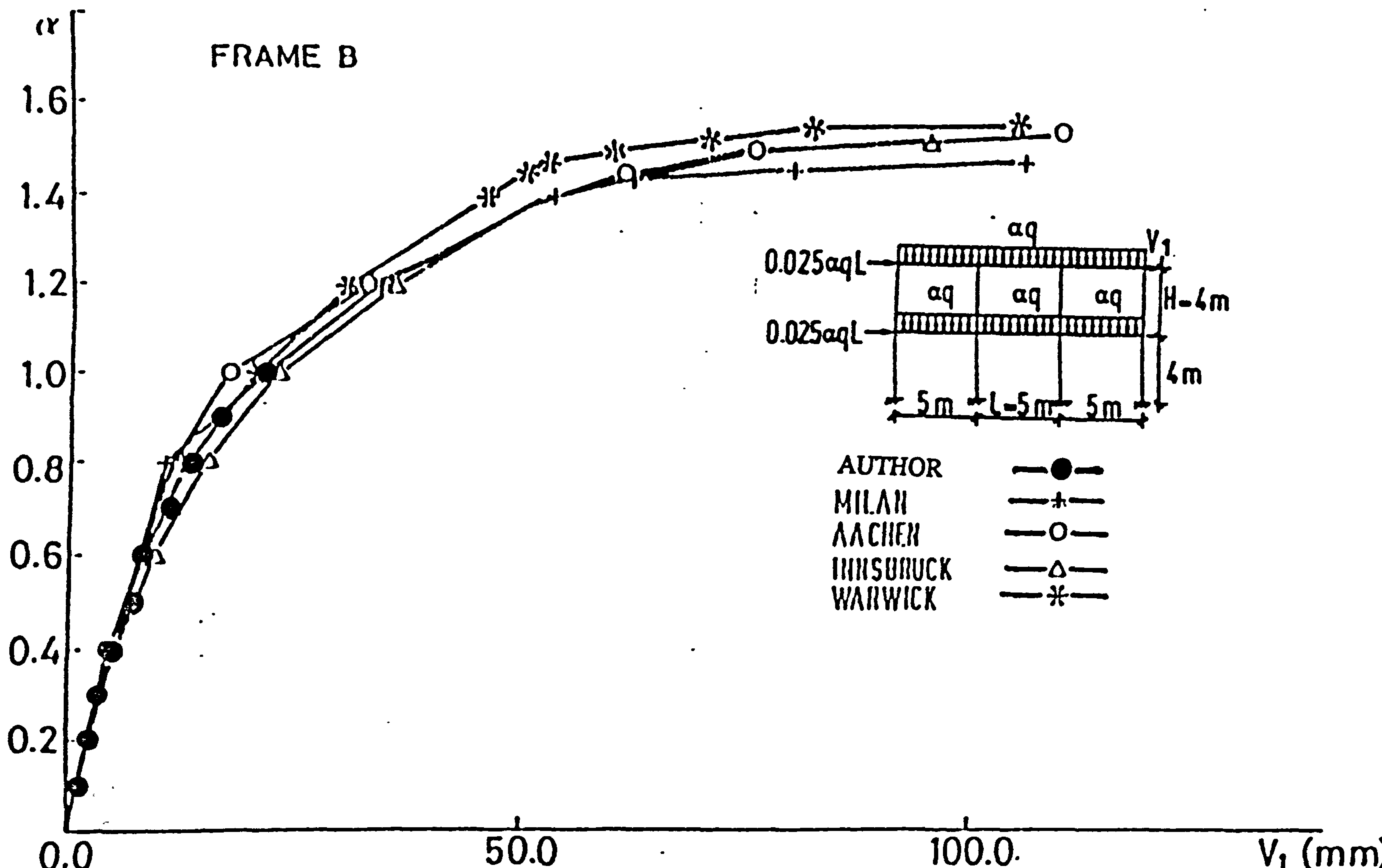


FIG.5-39 Comparison of different methods of analysis for Frame B.

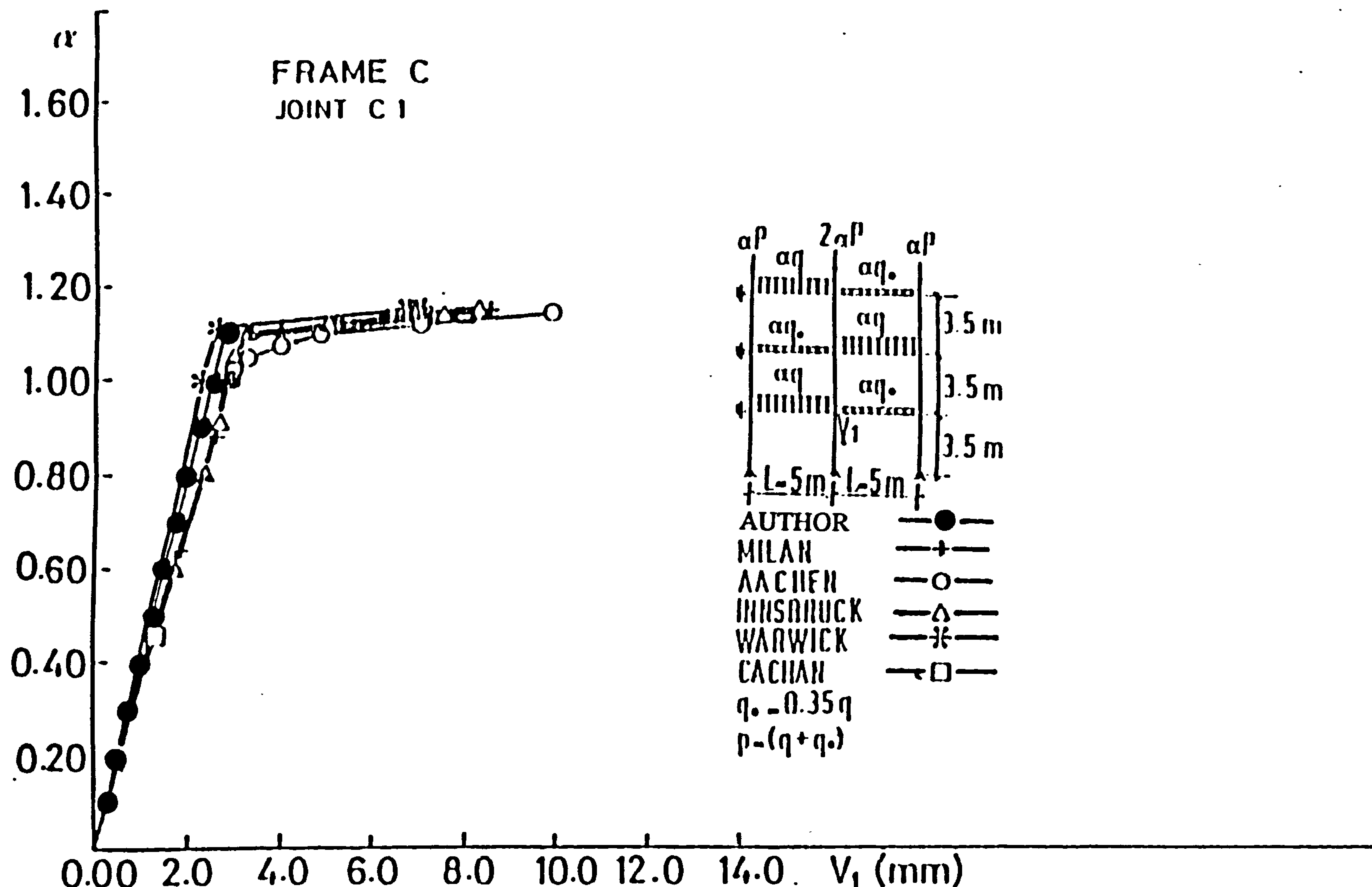


FIG.5-40 Comparison of different methods of analysis for Frame C.

CHAPTER VI
EFFECTS OF SEMI-RIGID JOINTS ON FRAME BEHAVIOUR AND CRITERIA FOR DESIGN

6.1 Introduction

Based on the available experimental data, beam-to-column connections are flexible, ranging in stiffness from those almost rigid to some types close to pinned. The characteristics of the joint are of importance in semi-rigid analysis and design only if the strength and/or stiffness of the connection between beams and columns are less than the strength and stiffness of the beams. For rigid and full strength connections the designer has only to take into account the moment-rotation relations of the beams and columns and therefore it is not necessary to incorporate joint characteristics in the computation. Thus an investigation into the influence of the joint characteristics on the structural response of frame is necessary only if semi-rigid or partial strength are used.

The influence of joint characteristics on structural response of frames is explored in this chapter by studying a number of different frames with various beam-to-column connections. The frames are analysed using the computer program developed by the author and described in the pevious chapter to investigate:

a) the effects of connection flexibility on the behaviour of unbraced frames, in particular:

   i) the contribution of the connection flexibility to the overall frame deformation, and

ii) the effects of the connection flexibility on the distribution of internal forces and moments in beams and columns;

b) the sensitivity of the results of frame analysis of complete frameworks, including second-order effects, to M-Φ characteristics;

c) pilot studies on:

i) an approximate method for ultimate load analysis and

ii) a criterion for first order analysis;

d) governing design criterion for semi-rigid unbraced frames.

6.2 Frame data:

Fourteen different frames were studied with various beam-to-column connections. The first nine frames are summarised in Table 6.1. The range of sections used within each frame is given and the ratio of total vertical load to total horizontal load (V/H) is stated. As the program could only deal with point loads, the uniformly distributed loads were simulated as point loads. The loading was considered in this study as three point loads, one at the centre of the beam and one at each end of it. The simulation of distributed loading by point loads was checked already in Chapter 5 by analysing Frames A, B and C. It was found that the difference affected the results only marginally, so it was concluded that the analysis was not sensitive to exact form in which the beam load acts on the frame. Frames A, B and E1-E7 employed extended end-plates, without column stiffeners. The M-Φ data for Frames A and B was derived by Tschemmernegg and Humer [6.1] for frames specified by Zandonini [6.2] as part of an ECCS study. The descriptions of Frames A and B with the complete results of semi-rigid analysis are given in Figures 5.26, 5.27, 5.28, 5.30b, 5.31a and 5.32a for both 1st and 2nd order analysis. Frames E1-E7 were designed during studies on the wind-connection method by Anderson et al [6.3], the connection stiffness being obtained from the proposals of the modified Frye and Morris [6.4] prediction equation (equ. (4.11)).

Frame configurations and loadings are shown in Figure 6.1 to 6.7 for Frames E1 to E7 respectively. The thickness of the end-plates in Frames E1-E7 varied from 6 mm to 28 mm, dependent mainly on the bending moment due to wind loading that had to be transmitted by the connection. All the members are in steel of grade 43 and the Young's modulus of elasticity was taken as E = 205 kN/mm².

The remaining five frames, designated H1-H5, were similar to Frame A except that header plate connections were used to obtain very flexible frames, devised to extend the range of the study. Again Frye and Morris's work [6.4] was used to determine the connection rotation (equ. (2.18) and (2.19)). The depth of the end-plate varied from 240 mm to 180 mm, and the thickness of the end-plate from 12 mm to 8 mm. Details of the connections are given in Figure 6.8.

6.3 Extent and significance of sway and distribution of bending moment from joint flexibility.

In order to make a comparison between rigid and semi-rigid analysis, all the frames were re-analysed assuming full rigid connections. The comparison of the results obtained for Frames A,B and E1 to E7 are presented in Table 6.2. The results of Frames A and B correspond to type 1 loading.

The effect of semi-rigid connections on sway deflection is shown in Figures 6.9 and 6.10 for Frames A and B respectively. The overall lateral displacement for the nine frames is shown in Table 6.2 and denoted by δ. For the frames studied, the lateral displacement δ increased from 37% up to 80% in comparison to rigid analysis. The value 80% corresponds to Frame E2 which is designed for minimum vertical load and maximum wind load.

Although, different beam sections and different column sections are used within each frame in E1-E7, only the highest value in beam end moment, M_c, and in mid span moment, M_s, are given in Table 6.2. The effect of incorporating the connection deformation into frame behaviour is to reduce the moment M_c at the beam-to-column connections by 2% to 48% and to increase the mid span sagging moment, M_s, from 5% up to 37%. However, the effect of the semi-rigid connections is insignificant for the axial forces, denoted by N in Table 6.2.

As a result of the analyses of the unbraced semi-rigidly connected frames, the following conclusions are reached:

i) The connection rotation contributes to the overall frame deformations, in particular sway under lateral loads.

ii) The connection rotation will affect the distribution of internal forces and moments in beams and columns.

An analysis which neglects connection deformation may thus be unable to predict realistically stresses and deflections.

6.4 Effect of M-Φ relationship on the elastic analysis results

To test the sensitivity of a structure to variations in connection moment-rotation behaviour, a series of frames were analysed repeatedly. A set of single bay single storey frames, with flush end-plate connections to unstiffened columns, were designed so that the connections, beams and columns used correspond to the available isolated experimental test data reviewed in Chapter 3. The frames were designed under gravity loading only because usually such connections are used for braced frames. This was done at a load level "1.4 D.L. + 1.6 L.L". In addition Frame A described in Chapter 5 was re-analysed in order to examine the sensitivity of results to change in M-Φ relationships. Only results of three frames are presented here, Frames S1, S2 and A. Frame configurations and loading conditions are shown in Figure 6.11. Connections used in Frames S_1 and S_2 are flush end-plate connections corresponding to test 17 and test 3 of Ostrander [6.5] respectively. Complete details of the connections are given in Appendix B of this thesis.

In Figures 6.12 to 6.14, the Y axis represents the percentage of connection stiffness taken in the frame analysis relative to connection stiffness given by the experimental result, ranging from 0% to 100% stiffer and from 0% to 100% more flexible. The X axis in Figures 6.12 and 6.13 for frames S1 and S2 respectively represents the percentage error in beam deflection, mid span moment and beam end moment relative to the corresponding analysis using the experimental M-Φ data. Percentage of error in lateral deflection at the top of Frame A is also presented in Figure 6.14. Examination of the analysis results has produced the following findings:

i) If the connection flexibility is overestimated, the beam deflection, lateral frame deflection and mid span moment are overestimated. However the beam end moments are underestimated. This is simply because a flexible connection attracts less moment. For a stiffer connection the opposite results take place.

ii) The wide variations in connection stiffness affected the beam deflection, mid span moment and the beam end moment by less than 6% for connection stiffness ranging from -30% to +30% (where the minus sign indicates a more flexible connection). However, for lateral frame deflection, the result is significant and it is of the order of 10% for a 30% increase or a decrease in connection stiffness.

Another set of Frames, F1 to F6, were designed under gravity loading only because such connections are usually used for braced frames. Frames F1 to F6 were analysed to examine the sensitivity of the predicted equation of M-Φ relationship given by equ. (3.24) for the unstiffened flush end-plate connections. Frame details are given in Table 6.3. The M-Φ data corresponding to each frame connection are given in Appendix B. The analysis results are shown in Tables 6.4 to 6.9. The frames were analysed at different load levels, relative to the design load "1.4 D.L.+ 1.6 L.L." (gravity loading only). The difference between using predicted M-Φ relationship (equ. (3.24)) and experimental in the analysis results lies within the range -20% to +20% (where negative sign indicates a more flexible connection). This over-estimatation or under-estimation of connection stiffness affects the elastic analysis of the structure by less than 5% in comparison to the results using the experimental M-Φ data. In view of the complexities involved in predicting connection behaviour, these above comparisons show that the M-Φ relationship developed by the present author for unstiffened flush end-plate connection is considered to be adequate for a connection rotation less than 0.02 radians.

6.5 Pilot studies on an approximate method for ultimate load analysis and criterion for first order analysis

For rigid frames, if the elastic critical load λ_{cr} is greater or equal to ten times the design load, the second order effects are assumed to be negligible and first order analysis can be used. The adequacy of this criterion for semi-rigid frames has been investigated using the program described in Chapter 5. For this task, the elastic critical load has to be determined.

In addition, to study the governing design criterion for semi-rigid unbraced frames, the ultimate collapse load has to be investigated.

6.5.1 Elastic critical load factor, λ_{cr}

The elastic critical load factor, λ_{cr}, of a frame is defined as the ratio by which each of the factored loads would have to be increased to cause elastic instability [6.7].

For frames with rigid and full strength connections, the lowest elastic critical load, λ_{cr}, is determined in this study by using a second order elastic analysis program in conjunction with a modified Southwell plot as shown for example in Figure 6.15. An estimation of λ_{cr} is carried out by extrapolating several positions on the plot. These positions are characterised by large horizontal deflections. It should be noted that when the analysis shows that the frame sways back into the horizontal load, then λ is greater than λ_{cr}. The dead and imposed factored loads are used as the basis for calculating λ_{cr} (i.e. 1.4 D.L. + 1.6 L.L.). These loads are applied as vertical point forces at the heads of the columns, coupled with a small horizontal disturbing force applied at roof level. The accuracy of this method was checked by Lok [6.8] by recalculating several values of λ_{cr} using the charts due to Wood [6.9]. In addition it is shown in the same reference [6.8] that the approximate analysis of Williams [6.10] is in good agreement with this above described method.

For frames with semi-rigid connections, the lowest elastic critical load factor, λ_{cr}, has been determined by applying vertical loads as point loads at the heads of the columns coupled with a small horizontal disturbing force of about 1% of the vertical load applied at the top of the frame as described above for rigid frames. The analysis was repeated at increasing levels of loading, a modified Southwell plot being used to determine the critical load. As the latter relates a bifurcation type buckling problem, the semi-rigid connections will consequently be characterised by their initial stiffnesses k_i.

A typical plot to determine λ_{cr} is shown in Figure 6.15 for Frame A. The loading of the frame shown in Figure 5.26b was regarded as corresponding to a load level

λ of unity. The elastic critical load level λ_{cr} was then calculated in the manner previously described. The value of 11.4 indicates that the critical load level is 11.4 times the level of loading shown in Figure 5.26b. The value of λ_{cr} with rigid joints was found to be 16.7 times the level of loading given in Figure 5.26b.

6.5.2 Approximate methods for ultimate load analysis

Although sophisticated computer programs are available to determine accurately the failure load, λ_f, of a rigidly connected structure, an approximate method of determining the failure load is an attractive alternative. Such an approach may also be required to provide a check on a computer method and to satisfy the engineer who wishes to maintain full control of the design process.

It is well known that the collapse load of an unbraced steel frame with rigid joints and subject to proportionally increasing loads may be obtained with reasonable accuracy by means of the Merchant-Rankine formula. This is similar to the well known empirical formula for individual members. Merchant [6.11] concluded that a simple interaction relationship provided the best result and postulated that the collapse load would be estimated by the following expression:

$$\frac{1}{\lambda_{f,mr}} = \frac{1}{\lambda_{cr}} + \frac{1}{\lambda_p} \qquad (6.1)$$

The above formula relates the failure load, denoted here by $\lambda_{f,mr}$ to the lowest elastic critical load factor, λ_{cr} and the rigid-plastic collapse load, λ_p. All these factors in equ. (6.1) are in fact multipliers of reference loads.

To take account of the beneficial effect of strain hardening and minimal composite action, Wood [6.9] modified this formula (equ. (6.1)) slightly by introducing a numerical factor 0.9 into the term dealing with the first order plastic hinge theory.

Therefore, the modified Merchant-Rankine formula becomes:

$$\frac{1}{\lambda_{f,mrw}} = \frac{1}{\lambda_{cr}} + \frac{0.9}{\lambda_p} \qquad \text{when } 4 \leqslant \frac{\lambda_{cr}}{\lambda_p} < 10 \qquad (6.2a)$$

and

$$\lambda_{f,mwr} = \lambda_p \qquad \text{when } \frac{\lambda_{cr}}{\lambda_p} \geqslant 10 \qquad (6.2b)$$

Frames with $\lambda_{cr}/\lambda_p < 4$ cannot be designed by equ. (6.2) and a more accurate method than the above formula should be used.

The use of the modified Merchant-Rankine formula requires the computation of two quantities:

i) the elastic critical load factor, λ_{cr} and

ii) the idealised rigid-plastic collapse load factor, λ_p.

The validation of equ. (6.2), when extended to unbraced steel frames with semi-rigid joints will be checked in this section.

The determination of the elastic critical load factor, λ_{cr}, for frames with either rigid or semi-rigid joints is given in the previous section (section 6.5.2).

However the rigid plastic collapse load factor, λ_p, is calculated by re-running the elasto-plastic analysis program [6.12] with Young's modulus of elasticity given a very high value. λ_p can also be obtained by the same analysis program with all the stability functions φ's given unit value. For frames with semi-rigid joints, the rigid plastic load factor, $\lambda_{p,author}$ is approximated by the present author by assuming the semi-rigid connection as a member of small length of the order of 5 cm with its plastic moment, M_p, equal to the maximum bending moment offered by the connection. It is the maximum bending moment given by the experimental M-Φ data. For frames where M-Φ data are obtained by Frye and Morris [6.4] prediction equations, such equations do not provide the moment capacity of the connection; a value of plastic moment of connection is taken as the plastic moment of the beam section connected to it.

To check the validity of equations (6.1) and (6.2) for frames with either rigid or semi-rigid joints, an accurate failure load is needed to be computed. Again the second-order elasto-plastic analysis [6.12] is used to determine the exact failure load, $\lambda_{f,exact}$, for frames with rigid joints and Ohta's analysis program [6.13] for frames with semi-rigid joints.

The results of the study of Frames A,B and E1-E7 are summarised in Table 6.10. The failure load from equations (6.1) and (6.2) are given under the heading $\lambda_{f,mr}$ and $\lambda_{f,mrw}$ respectively. The factor $\lambda_{p,author}$ corresponds to the predicted rigid plastic collapse load factor for semi-rigid frames. Table 6.10 also indicates the rigid-plastic collapse mechanisms, denoted by:

- B for simple beam-type collapse mechanism.
- C for combined mechanism.
- S for column sway mechanism.

Comparison of rigid and semi-rigid analysis results obtained for the nine frames presented in Table 6.10 indicates that:

i) the elastic critical load factor, λ_{cr}, is reduced significantly, by the order of 13.5 to 31.8 per cent in comparison to rigid analysis,

ii) the reduction in collapse load factor, $\lambda_{f,exact}$, is less pronounced in comparison to the reduction of the elastic critical load factor and it is of the order of 1.9 to 13.9 per cent.

Again, collapse load factor predicted by equ. (6.1), $\lambda_{f,mr}$, and the collapse load factor given by equ. (6.2), $\lambda_{f,mrw}$, are based on the calculated factors λ_{cr} and $\lambda_{p,author}$. A comparison of $\lambda_{f,mrw}$ and $\lambda_{f,exact}$ indicates the following results:

i) For Frame E2, λ_{cr}/λ_p is greater than 10 and consequently $\lambda_{f,mrw}$ will be given by λ_p. The error in predicting the collapse load is overestimated by 6 per cent in comparison to the exact value.

ii) For frames with $4 \leqslant \lambda_{cr}/\lambda_p < 10$ and where the rigid-plastic collapse mode is a combined mechanism, the modified Merchant-Rankine formula predicts satisfactory the collapse load factor. The error is less than 5.2% in comparison to $\lambda_{f,exact}$.

iii) For frames with $\lambda_{cr}/\lambda_p < 4$, the collapse load cannot be determined by $\lambda_{f,mr}$ or $\lambda_{f,mrw}$ and a more accurate method than the above equations (equ. (6.1) and (6.2)) should be used.

Recently, Jaspart [6.14, 6.15] investigated the extent of the Merchant-Rankine formula for the assessment of the ultimate load of frames with semi-rigid joints.
The elastic critical load factor, λ_{cr}, the plastic load factor, λ_p, and the collapse load factor, λ_f, for Frame A given in reference [6.14] are in excellent agreement with the present author result.

Based on comparison of simple procedure suggested for the computation of the critical and first order plastic load factor with the numerical simulation conducted by a non linear finite element analysis program on several semi-rigid frames, the author [6,15] concluded that:

i) Due to range of validity of the modified Merchant-Rankine formula (equ. 6.2a), even an appreciable error on the value of the elastic critical load factor λ_{cr} affects only slightly the value of the collapse multiplier λ_f. Therefore a larger freedom is offered when choosing the method aimed at assessing λ_{cr}.

However, any error made when assessing the plastic multiplier λ_p will result in a closely similar error on the collapse load, when the latter is derived from the use of the modified Merchant-Rankine formula. Therefore an accurate value of the plastic multiplier λ_p is required.

ii) The collapse load factor given by equ. (6.2a) is very accurate as far as the collapse mechanism is a combined mechanism. However, it is slightly conservative when a partial beam mechanism is governing and it is generally largely unsafe when a panel mechanism is commanding [6.15].

The generalisation and the range of validity of the modified Merchant-Rankine formula to frames with semi-rigid connections proved by Jaspart is in good agreement with the present author's findings.

6.5.3 Criterion for first order analysis

In several design documents [e.g. 6.16] it is stated that for rigid frames, if the elastic critical load λ_{cr} is greater or equal to ten times the design load, the second order effects are assumed to be negligible and first order analysis can be used. The adequacy of this criterion for frames with semi-rigid connections has been investigated using the second order frame analysis computer program [6.17] for the fourteen different frames described in section 6.2 of this chapter.

The results of the study are summarised in Tables 6.11. To measure the stiffness of the semi-rigid structure, the term "degree of flexibility" is defined as:

$$\{[\lambda_{cr\ rigid} - \lambda_{cr\ semi\text{-}rigid}] / \lambda_{cr\ rigid}\} \times 100\% \qquad (6.3)$$

where $\lambda_{cr\ rigid}$ is the elastic critical load factor with all joints taken as rigid; $\lambda_{cr\ semi\text{-}rigid}$ is the elastic critical load factor taking account of the initial stiffness of all semi-rigid connections. The calculation of the elastic critical load factor is described in section 6.5.1. In Table 6.11, the frame stiffnesses are expressed by the "degree of flexibility" defined by equation 6.3. ϵ_1 and ϵ_2 represent the maximum difference between first-order and second-order analysis for sway and bending moment respectively, expressed as a percentage of the second-order result. It should be noted that ϵ_2 is calculated for those moments that would be critical for design. Comparison of bending moments in the windward leg of Frame A (Figures 5.27 and 5.28) shows some large percentage differences between first-order and second-order analysis, but the magnitude of these moments are small. As sections would be chosen on the basis of the much larger moments in the leeward leg, it is the latter which are used to calculate ϵ_2.

To investigate the criterion for neglect of second-order effects, the semi-rigid frames was then analysed at a load level corresponding to one-tenth of the critical load for the frame - in the case of Frame A, at loads 1.14 times those shown in Figure 5.30b. Both first-order and second-order analyses were performed and the results compared. The differences were expressed in terms of ϵ_1 and ϵ_2 defined above. For the semi-rigid Frame A, it is seen that at one-tenth of the critical load the neglect of second-order effects underestimates the bending moments by no more than 8% and sway deflection by 12%.

For Frames A and B, λ = 1.0 corresponds to the loads specified by Zandonini [6.2].

Frames E1-E7 were designed in UB and UC sections using realistic 'characteristic' values of loading for the United Kingdom (British Standards Institution [6.18, 6.19]). When dead, imposed and wind loading were combined, British practice is to ensure that the structure can withstand "design loads" equal to 1.2 times the characteristics loads (British Standards Institution [6.7]). The load level denoted in this study corresponded to these "design loads".

As Frames H1-H5 were derived from Frame A, the load level $\lambda = 1.0$ corresponds to the load shown in Figure 6.8a.

The general trend in Frames E1-E7 observed from Table 6.1 and Table 6.11 is for the critical load to increase as the ratio V/H falls (compare for example Frames E3 and E5). Frames designed to withstand relatively high horizontal forces had greater lateral stiffness, which in turn gave higher values for the critical load. For Frames E2 and E5, it was not possible to analyse at one-tenth of the semi-rigid critical load. This was because this load level was far above that for which the frame had been designed and one or more connections were incapable of withstanding the higher loading. For this reason, it was decided to analyse also Frames E1-E7 (and also Frames A and B) at the load level $\lambda = 1.0$. It will be recalled that for Frames E1-E7 this corresponds to the 'design' load for the ultimate limit state under combined loading.

Difficulties similar to those encountered in Frames E2 and E5 also arose in some other of the Frames H1-H5. These frames had been devised without design calculations and possessed very flexible semi-rigid connections. The manner in which Frames H1-H5 were devised causes the load level $\lambda = 1.0$ to be without significance and results are not therefore presented for this situation.

From the results in Table 6.11 it is apparent that when the elastic critical load for semi-rigidly connected frames is greater than or equal to ten times the design load the second order effects are not significant. Unless the frame is very flexible the maximum error in neglecting the second-order effect is 12% for both sway and bending moment, except for frames in which the degree of flexibility is greater than 50%.

6.6 Governing design criterion for semi-rigid unbraced frames

For Frames A,B and E1-E7, the ultimate collapse load has been calculated by Ohta's computer analysis program [6.13]. The load level at which the second-order overall sway index equalled 1/300 was also calculated by the present author assuming elastic behaviour. This is termed the "h/300 load". By comparing these two load levels, an insight can be gained into the likelihood of sway controlling design. The results are given in Table 6.12.

In design to BS5950 [6.7] the partial safety factor on combined load for the ultimate limit state is 1.2 whilst the limiting sway index of 1/300 applies to unfactored loads. This partial safety factor may be taken as 1.35 in design to EC3 [6.20]. Thus those frames for which the collapse load exceeded 1.2 times the "h/300" load would be governed by deflection according to BS5950. It can be seen from Table 6.12 that this is the case for all the frames studied in this way, except for Frame E6. This four-bay frame was subjected to very light wind loading, as indicated by V/H in Table 6.1.

6.7 Conclusions

Studies carried out on practical multi-storey, unbraced frames have investigated the extent and significance of sway and distribution of bending moment from joint flexibility. In addition, these frames have enabled guidance to be given on the relative influence of sway deflection and ultimate strength as design criteria under combined loading. These frames have been subjected to realistic values of vertical and horizontal loads that are normally encountered in practice.

Connection flexibility has significant effects on frame behaviour. For realistic predictions, it should be included in analysis and design. Internal member forces may be strongly affected by connection rotations. The above frame investigations have shown that neglect of connection rotation may lead to serious underestimation of frame deflection.

The studies carried out on single storey-one bay frames under gravity loading have shown that the wide variations in connection stiffness affected the analysis results by less than 6 per cent for an overestimation or an underestimation of connection rotation up to 30 per cent of the true value.

The studies of multi-storey unbraced frames have also shown the modified Merchant-Rankine formula, given by equation (6.2a) provided a close estimation of the non-linear elasto-plastic failure load, λ_f, for frames with $4 \leqslant \lambda_{cr}/\lambda_p < 10$ and where the rigid-plastic collapse mode is a combined mechanism.

A parameter entitled "degree of flexibility" has been introduced as a measure of the effect of semi-rigid joints on the stiffness of a frame. This parameter offers a useful criteria for assessing the importance of connection flexibility.

It has been demonstrated that the errors in sway deflection and bending moment arising from neglect of second-order effects are unlikely to be significant if the semi-rigid elastic critical load exceeds ten times the design load and the degree of flexibility is less than 50%.

Even for frames with low degrees of flexibility it is likely that under combined loading the serviceability limit on sway will control design rather than ultimate strength.

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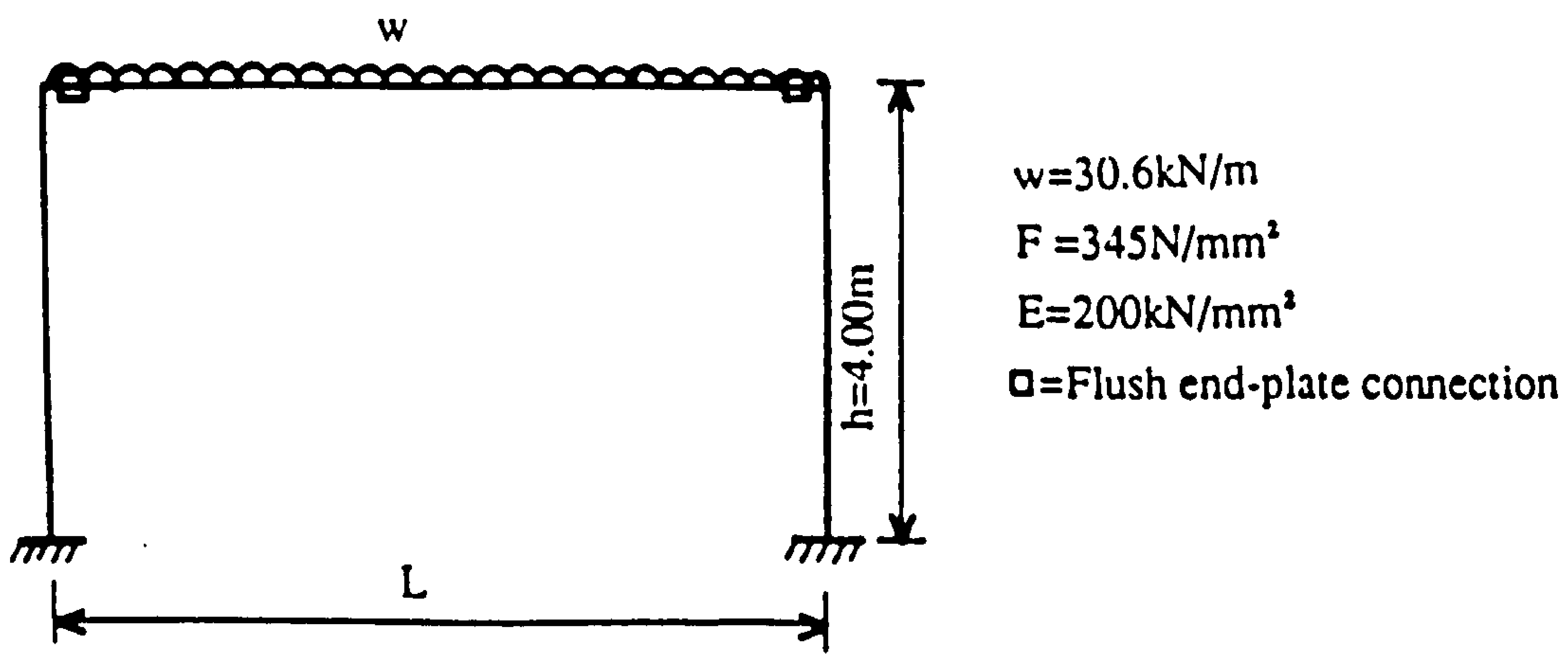
Ref. No.	Frame	Bay width (m)	Storey height (m)	$\frac{V}{H}$	Top storey			Bottom Storey		
					Beam	Ext Column	Int. Column	Beam	Ext Column	Int. Column
A	3 storey 1 bay	5.0	4.0	18.6	IPE300	HE200B	-	IPE300	HE200B	
B	2 storey 3 bay	5.0	4.0	85.7	IPE300	HE160A	HE160A	IPE300	HE160A	HE160A
E1	3 storey 1 bay	5.0	4.0	20.0	305x165x46 UB	203x203x46 UC	-	305x165x46 UB	203x203x46 UC	
E2	4 storey 1 bay	5.0	3.75	3.6	254x102x28 UB	203x203x46 UC	-	457x152x52 UB	254x254x73 UC	
E3	4 storey 1 bay	7.5	3.75	44.8	305x165x54 UB	152x152x30 UC	-	457x152x74 UB	203x203x46 UC	
E4	4 storey 1 bay	5.0	3.75	7.1	254x102x28 UB	152x152x37 UC	-	457x152x67 UB	203x203x60 UC	
E5	4 storey 1 bay	7.5	3.75	10.6	305x165x54 UB	203x203x46 UC	-	610x229x125 UB	203x203x86 UC	
E6	4 storey 4 bay	7.5	3.75	209	305x165x54 UB	152x152x30 UC	203x203x46 UC	457x152x74 UB	203x203x46 UC	203x203x60 UC
E7	7 storey 2 storey	5.0	3.75	6.6	254x102x28 UB	152x152x37 UC	203x203x46 UC	406x178x60 UB	254x254x89 UC	254x254x132 UC

Table 6.1 Summary of Frames A, B and E1-E7.

Frame Ref. No.	Analysis	M_e (kNm)	error (%)	M_s (kNm)	error (%)	N (kN)	error (%)	δ (mm)	error (%)
A	Rigid	97.8	-22%	83.4	+11%	338.1	-0.5%	32.6	+70%
	Semi-rigid	76.0		92.9		336.4		55.4	
B	Rigid	78.8	-48%	79.9	+24%	421.1	-3.5%	11.3	+57%
	Semi-rigid	40.9		98.9		406.0		17.7	
E1	Rigid	109.4	-18%	123.2	+5%	405.2	-0.2%	55.2	+55%
	Semi-rigid	90.0		129.6		404.4		85.5	
E2	Rigid	316.0	-7%	66.0	+22%	560.2	-1.1%	132.6	+80%
	Semi-rigid	292.2		80.4		553.8		238.6	
E3	Rigid	143.5	-22%	265.6	+8%	650.6	-0.1%	31.3	+66%
	Semi-rigid	111.8		285.7		651.2		52.1	
E4	Rigid	202.0	-7%	99.2	+7%	511.3	-0.2%	87.0	+37%
	Semi-rigid	187.0		106.1		510.0		119.1	
E5	Rigid	236.9	-7%	224.9	+9%	648.3	-0.1%	45.8	+36%
	Semi-rigid	219.8		245.1		647.7		62.1	
E6	Rigid	200.0	-44%	211.7	+30%	1315.9	-4%	10.7	+59%
	Semi-rigid	111.8		275.2		1262.6		17.0	
E7	Rigid	347.4	-2%	59.8	+37%	1208.7	-0.4%	250.8	+56%
	Semi-rigid	340.1		82.4		1204.4		391.8	

Table 6.2 Comparison of rigid and semi-rigid analysis results for Frame A, B and El-E7



a) Frame configuration

Frame Ref. No.	Beam section	Column section	Beam span L(m)	T_p (mm)	Connection type
F1	W12x27	W8x24	7.50	9.525	Test 17 [6.5)
F2	W10x21	W8x28	6.00	9.525	Test 3 [6.5]
F3	W12x27	W8x40	7.50	15.875	Test 13 [6.5]
F4	W12x27	W8x24	7.50	12.700	Test 18 [6.5]
F5	W10x21	W8x28	6.00	12.700	Test 1 [6.5]
F6	254x102 UB22	152x152 UC23	4.00	12.700	Test MJ/12 [6.6]

Table 6.3 Frames and connection details for Frames F1-F6

M-Φ data	Comparison of experimental and predicted connection stiffness	Load level in relation to design load	Connection rotation (x 10^{-3} radians)	Beam deflection (cm)	Mid span moment (kNm)	Beam end moment (kNm)
M-Φ exp	3.5% stiffer	0.80	5.737679	3.306	136.36	36.21
M-Φ pred			5.599939	3.291	135.98	36.59
M-Φ exp	4.8% stiffer	1.00	8.294272	4.263	173.60	42.23
M-Φ pred			8.043544	4.235	172.92	42.91
M-Φ exp	5.5% stiffer	1.20	11.562481	5.299	212.74	46.31
M-Φ pred			11.192386	5.258	211.79	47.32

Moment capacity of the connection is 65.31 kNm

At load level 1.20 the beam end moment is equal to 46.31 kNm which is about 71% of moment capacity of the connection.

Table 6.4 Analysis results for Frame Fl

M-Φ data	Comparison of experimental and predicted connection stiffness	Load level in relation to design load	Connection rotation (x 10^{-3} radians)	Beam deflection (cm)	Mid span moment (kNm)	Beam end moment (kNm)
M-Φ exp	4.5% flexible	0.80	5.260952	2.285	81.45	28.95
M-Φ pred			5.437733	2.304	81.84	28.57
M-Φ exp	3.8% flexible	1.00	7.551651	2.965	103.98	34.09
M-Φ pred			7.7714345	2.983	104.34	33.74
M-Φ exp	0.9% flexible	1.20	10.29400	3.695	127.51	38.26
M-Φ pred			10.389119	3.705	127.51	38.25
M-Φ exp	1.0% flexible	1.30	11.851837	4.080	139.68	39.94
M-Φ pred			11.923378	4.088	139.83	39.78
M-Φ exp	0.6% flexible	1.40	13.407602	4.466	151.85	41.62
M-Φ pred			13.459949	4.471	151.95	41.51

Moment capacity of the connection is 54.92 kNm

At load level 1.40 the beam end moment is equal to 41.62 kNm which is about 75% of moment capacity of the connection.

Table 6.5 Analysis results for Frame F2

M-Φ data	Comparison of experimental and predicted connection stiffness	Load level in relation to design load	Connection rotation (x 10^{-3} radians)	Beam deflection (cm)	Mid span moment (kNm)	Beam end moment (kNm)
M-Φ exp	13% stiffer	0.80	4.782073	2.845	125.61	47.01
M-Φ pred			4.357885	2.787	124.22	48.40
M-Φ exp	9.5% stiffer	1.00	6.532873	3.635	158.94	56.98
M-Φ pred			6.109387	3.577	157.56	58.38
M-Φ exp	8.3% stiffer	1.20	8.634091	4.473	193.46	65.82
M-Φ pred			8.160205	4.408	191.91	67.38
M-Φ exp	7.8% stiffer	1.30	9.848100	4.915	211.27	69.71
M-Φ pred			9.348723	4.847	209.63	71.36
M-Φ exp	7.4% stiffer	1.40	11.152987	5.369	229.38	73.30
M-Φ pred			10.629375	5.298	227.67	75.03

Moment capacity of the connection is 100.12 kNm

At load level 1.40 the beam end moment is equal to 73.30 kNm which is about 73% of moment capacity of the connection.

Table 6.6 Analysis results for Frame F3

M-Φ data	Comparison of experimental and predicted connection stiffness	Load level in relation to design load	Connection rotation (x 10^{-3} radians)	Beam deflection (cm)	Mid span moment (kNm)	Beam end moment (kNm)
M-Φ exp	15% stiffer	0.80	5.908472	3.326	136.82	35.75
M-Φ pred			5.348697	3.263	135.30	37.27
M-Φ exp	8% stiffer	1.00	8.116859	4.243	173.12	42.71
M-Φ pred			7.702724	4.196	172.00	43.84
M-Φ exp	3.6% stiffer	1.20	10.923528	10.923	211.07	48.05
M-Φ pred			10.685505	10.685	210.43	48.70
M-Φ exp	2% stiffer	1.30	12.335281	5.722	230.08	50.70
M-Φ pred			12.177419	5.704	229.66	51.13

Moment capacity of the connection is 76.27 kNm

At load level 1.30 the beam end moment is equal to 50.70 kNm which is about 67% of moment capacity of the connection.

Table 6.7 Analysis results for Frame F4

M-Φ data	Comparison of experimental and predicted connection stiffness	Load level in relation to design load	Connection rotation (x 10^{-3} radians)	Beam deflection (cm)	Mid span moment (kNm)	Beam end moment (kNm)
M-Φ exp	12% stiffer	0.80	4.633938	2.216	80.10	30.31
M-Φ pred			4.254456	2.175	79.27	31.14
M-Φ exp	16% stiffer	1.00	6.619080	2.863	101.97	36.12
M-Φ pred			5.938306	2.788	100.49	37.60
M-Φ exp	19% stiffer	1.20	8.993284	3.552	124.70	41.09
M-Φ pred			7.967942	3.440	122.48	43.31
M-Φ exp	19.5% stiffer	1.30	10.303248	3.911	136.33	43.31
M-Φ pred			9.125344	3.782	133.79	45.87
M-Φ exp	21% stiffer	1.40	11.735328	4.283	148.24	45.26
M-Φ pred			10.326829	4.129	145.20	48.32

Moment capacity of the connection is 61.92 kNm

At load level 1.40 the beam end moment is equal to 45.26 kNm which is about 73% of moment capacity of the connection.

Table 6.8 Analysis results for Frame F5

M-Φ data	Comparison of experimental and predicted connection stiffness	Load level in relation to design load	Connection rotation (x 10^{-3} radians)	Beam deflection (cm)	Mid span moment (kNm)	Beam end moment (kNm)
M-Φ exp	11.5% flexible	0.80	0.763696	0.791	38.65	10.34
M-Φ pred			0.853620	0.795	38.77	10.22
M-Φ exp	12.3% flexible	1.00	1.080161	0.995	48.50	12.74
M-Φ pred			1.214334	1.001	48.69	12.55
M-Φ exp	6.3% flexible	1.20	1.807697	1.219	58.93	14.57
M-Φ pred			1.715495	1.215	58.81	14.70
M-Φ exp	11.4% stiffer	1.30	2.186588	1.331	64.17	15.46
M-Φ pred			1.995191	1.323	63.91	15.72
M-Φ exp	20% stiffer	1.40	2.727624	1.452	69.63	16.13
M-Φ pred			2.336291	1.434	69.10	16.67

Moment capacity of the connection is 25.00 kNm

At load level 1.40 the beam end moment is equal to 16.13 kNm which is about 64% of moment capacity of the connection.

Table 6.9 Analysis results for Frame F6

Table 6.10 Failure load for Frames A, B and E1-E7

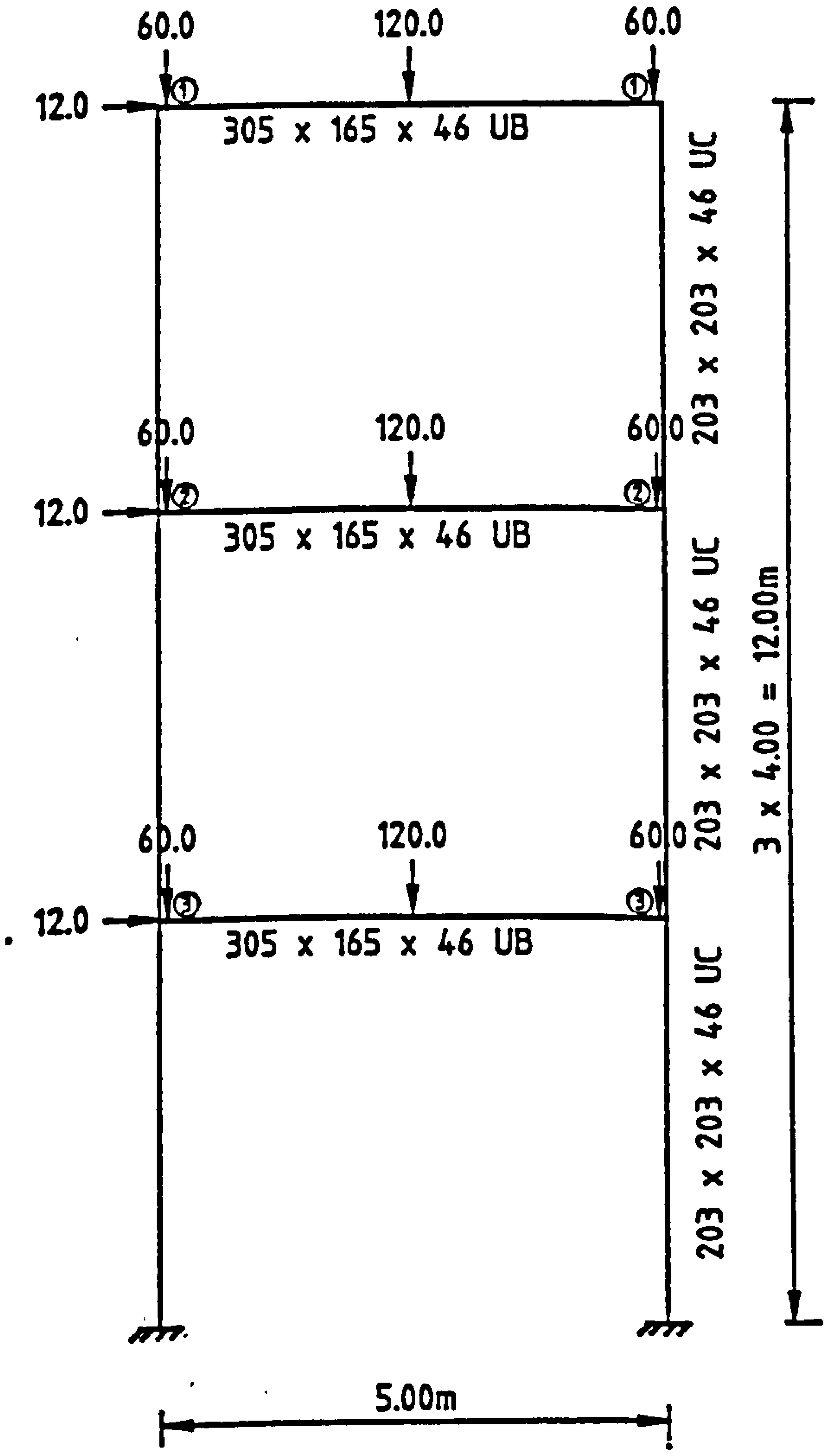
Frame Ref. No.	Rigid Analysis						Semi-Rigid Analysis						Type of mechanism for rigid plastic analysis
	λ_{cr}	λ_p	$\frac{\lambda_{cr}}{\lambda_p}$	$\lambda_{f,exact}$	$\lambda_{f,mr}$	$\lambda_{f,mrw}$	λ_{cr}	$\lambda_{p,author}$	$\frac{\lambda_{cr}}{\lambda_p}$	$\lambda_{f,exact}$	$\lambda_{f,mr}$	$\lambda_{f,mrw}$	
A	16.75	1.98	8.45	1.84	1.77	1.95	11.43	1.82	6.29	1.63	1.57	1.71	C
B	6.66	1.98	3.37	1.80	1.52	1.65	5.76	1.62	3.61	1.55	1.27	1.38	B
E1	9.24	1.74	5.32	1.60	1.46	1.60	6.88	1.74	3.96	1.57	1.39	1.51	B
E2	25.15	1.24	20.35	1.18	1.18	1.30	17.24	1.24	13.95	1.16	1.15	1.27	C
E3	8.44	1.58	5.34	1.49	1.33	1.45	6.22	1.58	3.93	1.44	1.26	1.37	C
E4	17.51	1.53	11.46	1.41	1.40	1.55	14.82	1.53	9.70	1.37	1.29	1.41	S
E5	22.60	1.94	11.67	1.81	1.78	1.96	19.52	1.94	10.08	1.72	1.76	1.94	S
E6	6.40	1.19	5.38	1.36	1.00	1.10	4.50	1.19	3.78	1.24	0.94	1.02	S
E7	12.24	1.15	10.66	1.00	1.05	1.15	9.48	1.15	8.29	1.08	1.02	1.12	C

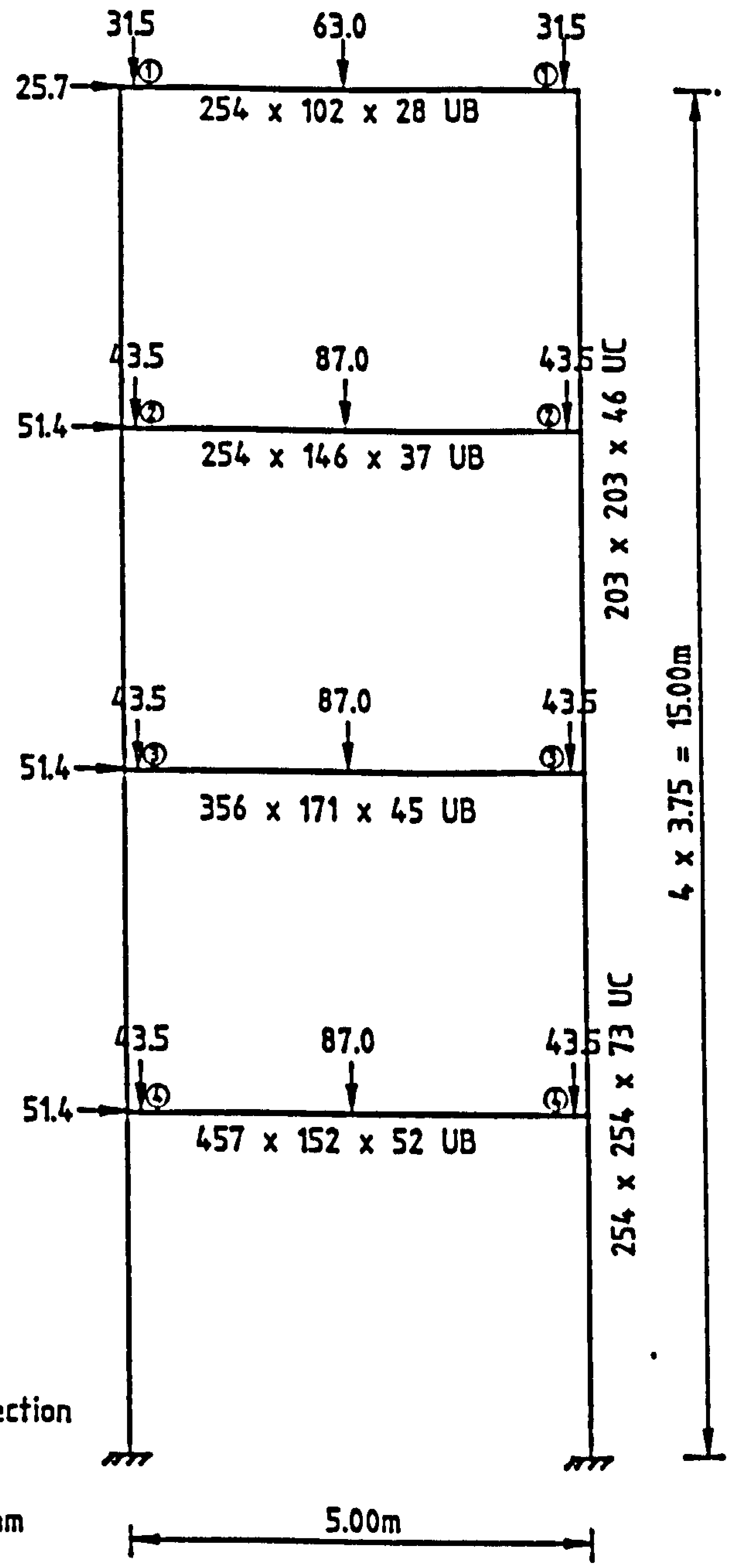
Frame Ref. No.	Semi-rigid λ_{cr}	Rigid λ_{cr}	Degree of Flexibility	Semi-rigid frames				
				$\lambda = \lambda_{cr}/10$			$\lambda = 1.0$	
				λ	ϵ_1	ϵ_2	ϵ_1	ϵ_2
A	11.4	16.7	32%	1.14	12%	8%	9%	6%
B	5.86	6.66	12%	0.59	9.1%	8%	18%	20%
E1	6.88	9.24	26%	0.69	9%	6%	15%	12%
E2	17.2	25.2	32%	1.72	-	-	9.1%	8.5%
E3	6.22	8.44	26%	0.62	9.5%	4.5%	16%	7%
E4	14.8	17.5	15%	1.48	9.6%	8.5%	7%	7%
E5	19.5	22.6	14%	1.95	-	-	5%	4%
E6	4.50	6.40	30%	0.45	9.5%	9%	23%	25%
E7	9.48	12.2	23%	0.95	10%	11.5%	11%	12%
H1	8.32	16.7	50%	0.83	13%	4.4%		
H2	7.12	16.7	57%	0.71	10.8%	7.8%		
H3	5.96	16.7	64%	0.60	13.5%	10%		
H4	5.18	16.7	69%	0.52	-	-		
H5	4.13	16.7	75%	0.41	-	-		

Table 6.11 Results of elastic analyses.

Frame Ref. No.	Collapse Load λ_f	Load level at h/300	Controlling design criterion
A	1.70	0.85	Sway deflection
B	1.55	0.85	
E1	1.57	0.52	
E2	1.16	0.26	
E3	1.44	1.00	
E4	1.37	0.45	
E5	1.72	0.80	
E6	1.24	> 1.24	Strength
E7	1.08	0.35	Sway deflection

Table 6.12 Governing design criterion.

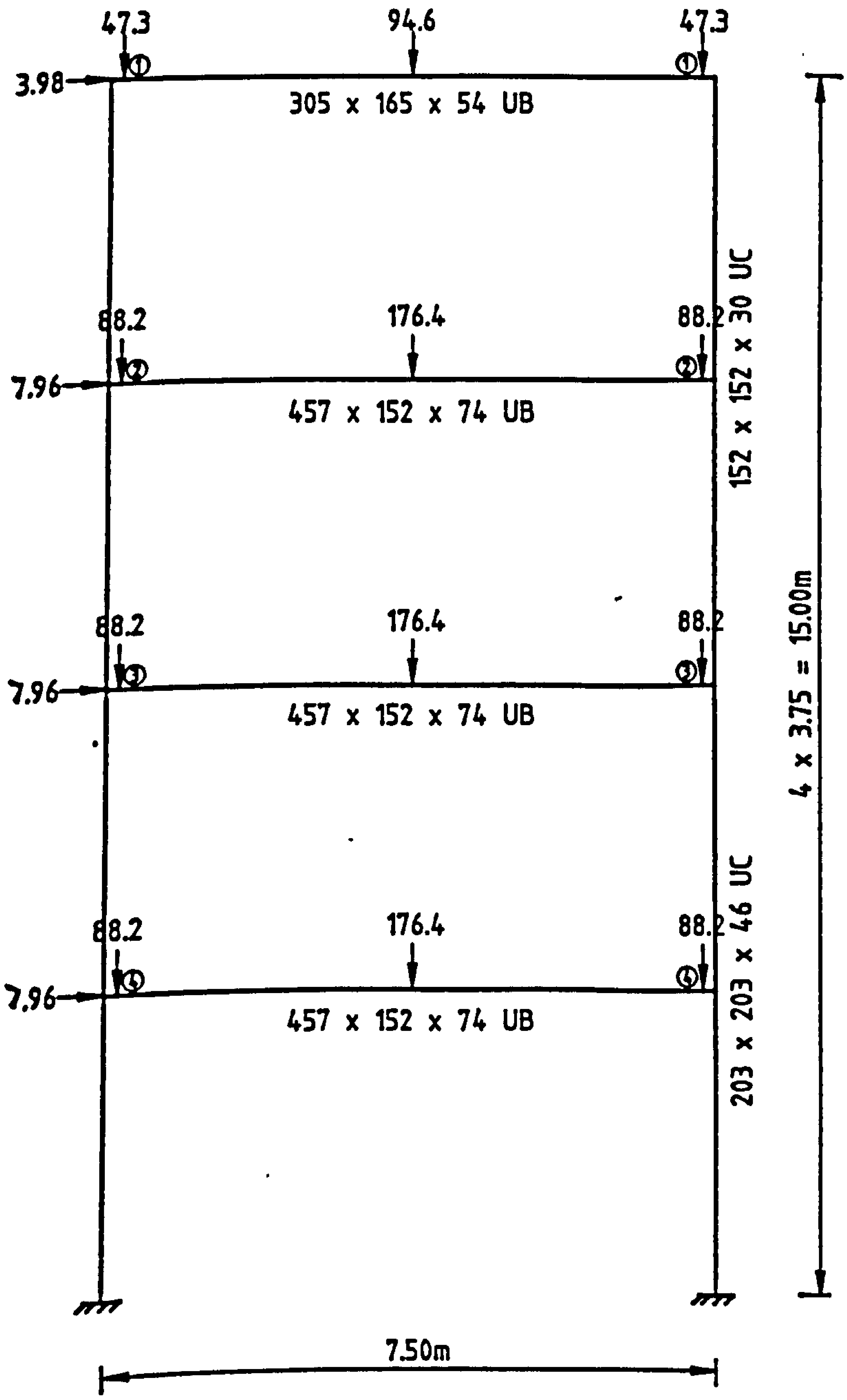


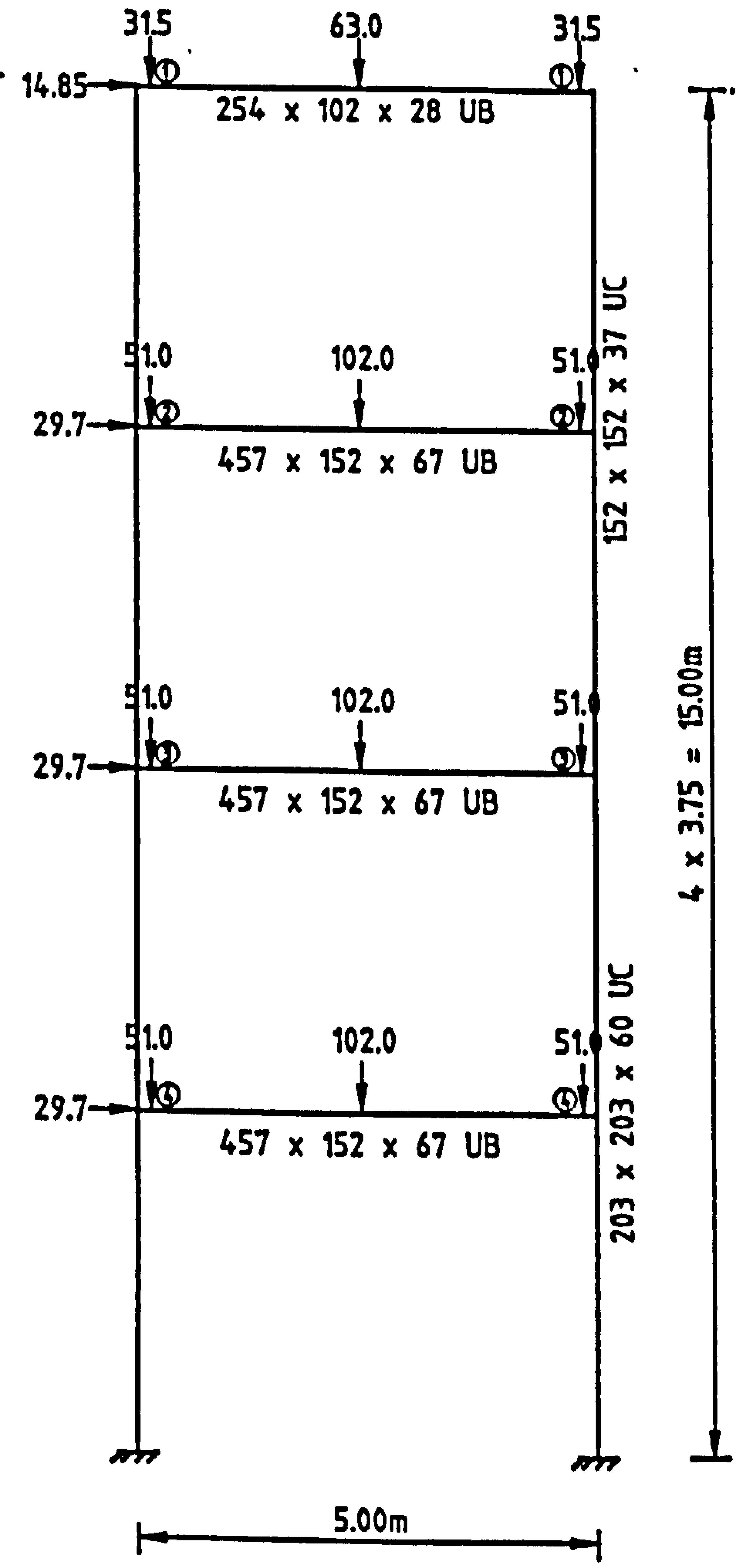


② Type of semi-rigid connection
All loads in kN
E=205kN/mm^2
Scale : Frame 1mm to 100mm

Figure 6.1
Three storey-one bay frame
"Frame E1"

Figure 6.2
Four storey-one bay frame
"Frame E2"

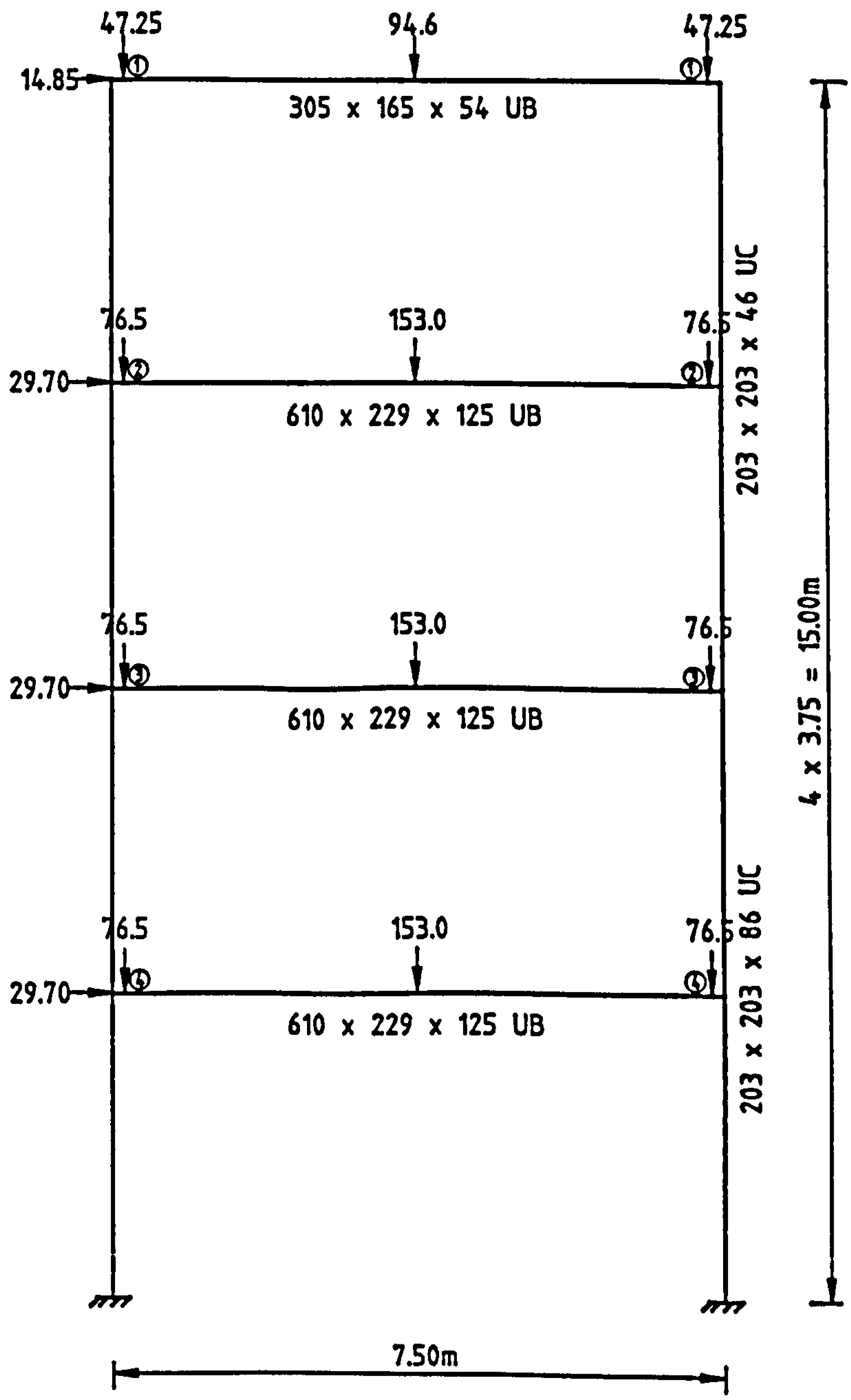




② Type of semi-rigid connection
All loads in kN
$E=205kN/mm^2$
Scale : Frame 1mm to 100mm

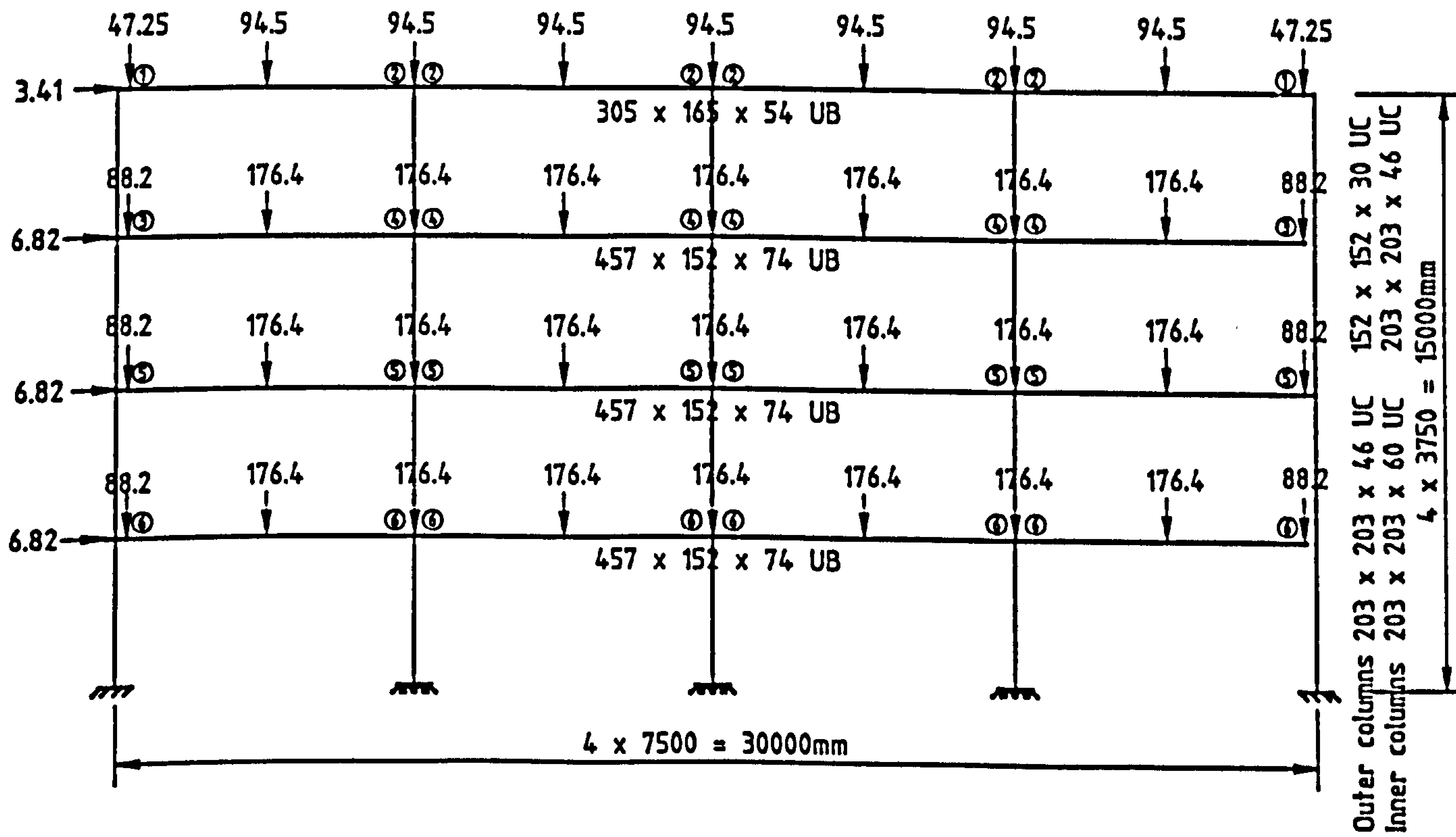
Figure 6.3
Four storey-one bay frame
"Frame E3"

Figure 6.4
Four storey-one bay frame
"Frame E4"



② Type of semi-rigid connection
All loads in kN
E=205kN/mm²
Scale : Frame 1mm to 100mm

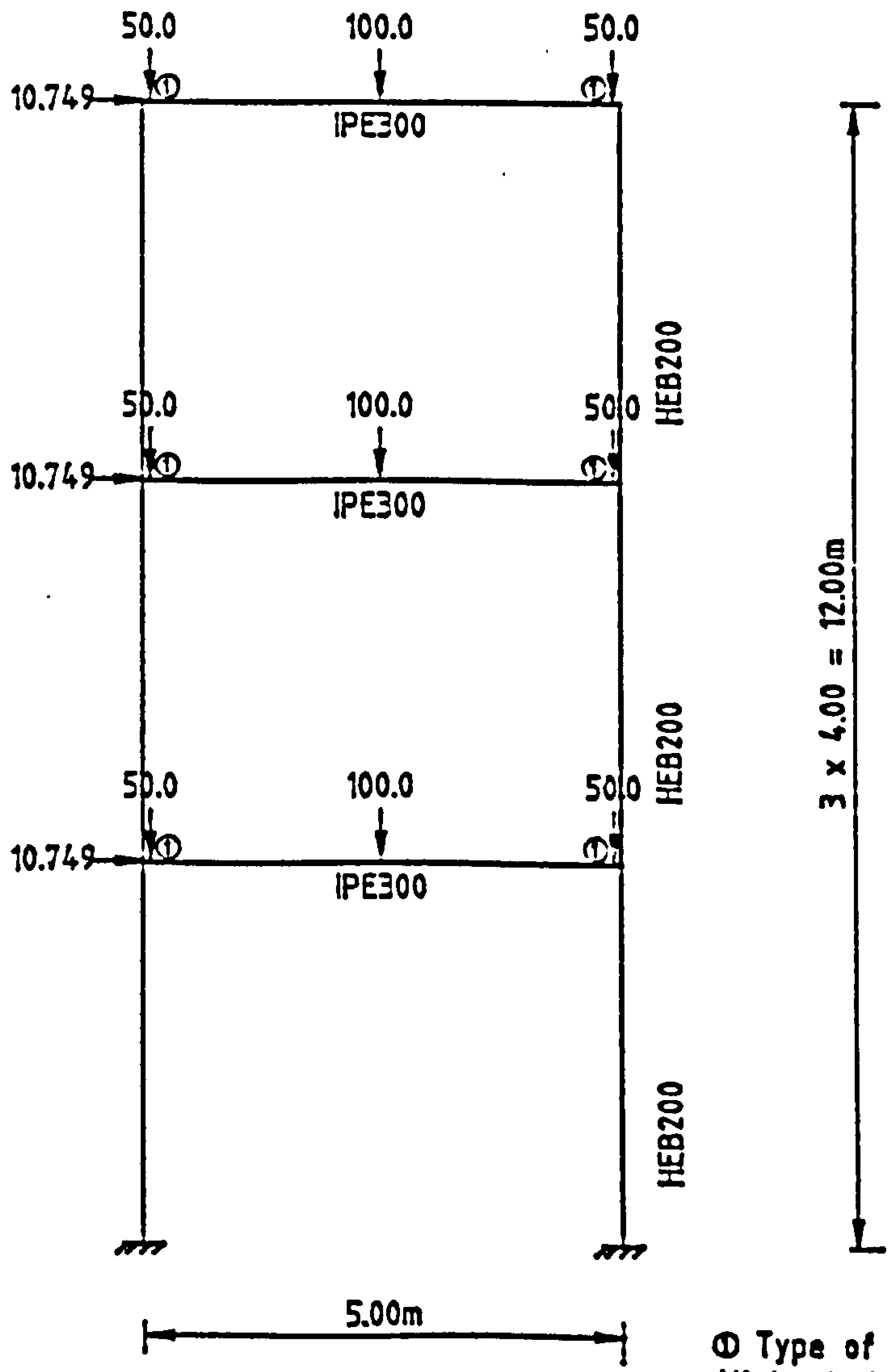
Figure 6.5
Four storey-one bay frame
"Frame E5"



② Type of semi-rigid connection
All loads in kN
E=205kN/mm²
Scale : Frame 1mm to 200mm

Figure 6.6
Four storey-four bay frame
"Frame E6"

Figure 6.7 Seven storey-two bay frame "Frame E7"



a) Frame configuration

① Type of semi-rigid connection
All loads in kN
$E=210kN/mm^2$
Scale : Frame 1mm to 100mm

Frame	End-plate thickness (mm)	Horizontal gauge distance (mm)	Length of plate (mm)	Beam web thickness (mm)
A1	12.0	100.0	240.0	7.1
A2	10.0	100.0	240.0	7.1
A3	8.0	100.0	240.0	7.1
A4	8.0	100.0	210.0	7.1
A5	8.0	100.0	180.0	7.1

Figure 6.8 Connection details for Frames H1, H2, H3, H4 and H5

	(1) Rigid joints 1st order	(2) 2nd order	(3) Semi-rigid joints 1st order	(4) 2nd order
Top storey level	$\frac{1}{576}$	$\frac{1}{552}$	$\frac{1}{305}$	$\frac{1}{279}$
Second storey level	$\frac{1}{326}$	$\frac{1}{307}$	$\frac{1}{191}$	$\frac{1}{172}$
Bottom storey level	$\frac{1}{346}$	$\frac{1}{325}$	$\frac{1}{247}$	$\frac{1}{224}$

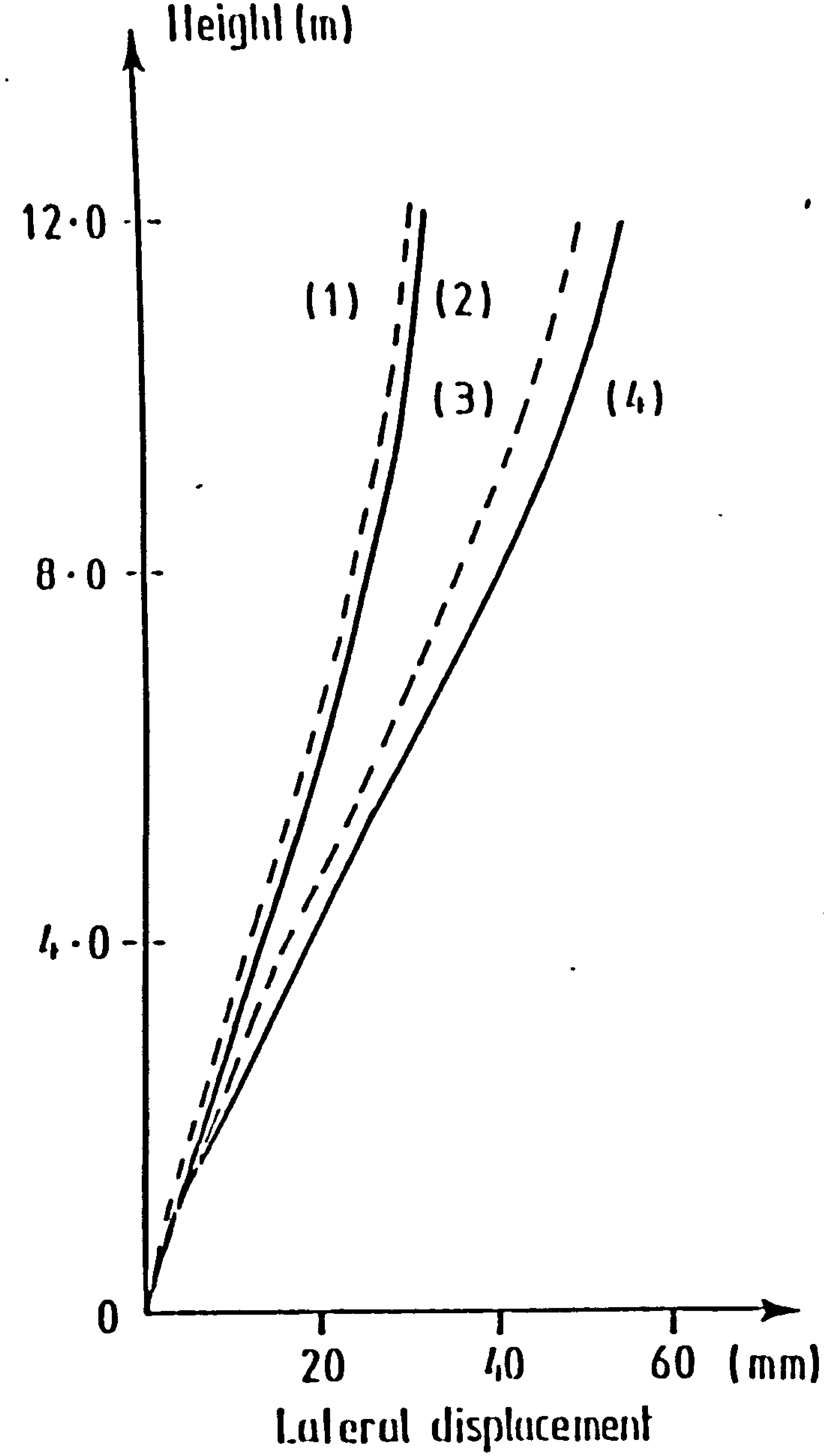


Figure 6.9 Effect of semi-rigid connections on sway deflection for Frame A, Type 1 loading.

	(1) Rigid joints 1st order	(2) 2nd order	(3) Semi-rigid joints 1st order	(4) 2nd order
Top storey level	$\frac{1}{1067}$	$\frac{1}{964}$	$\frac{1}{615}$	$\frac{1}{512}$
Bottom storey level	$\frac{1}{659}$	$\frac{1}{561}$	$\frac{1}{504}$	$\frac{1}{406}$

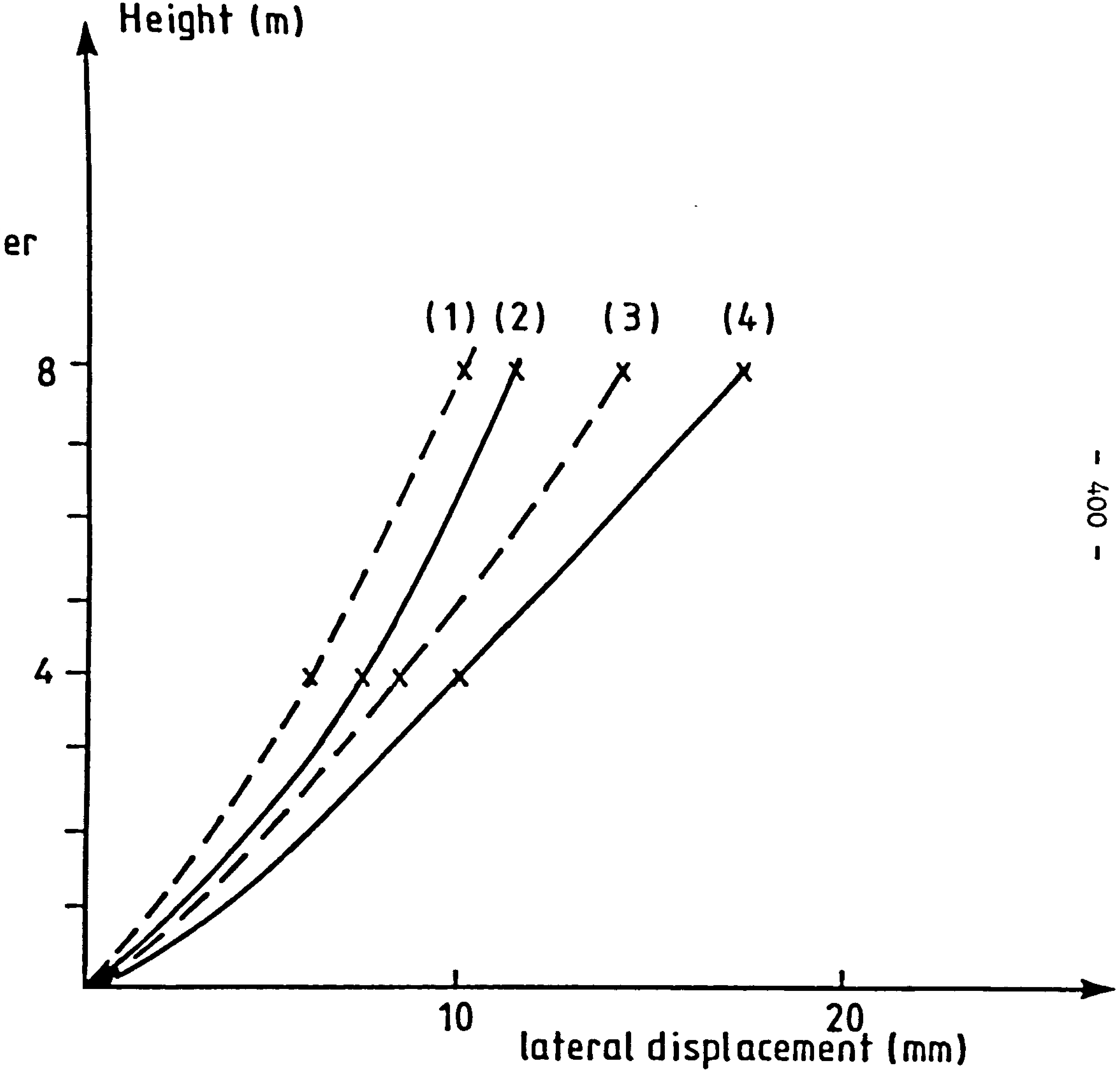
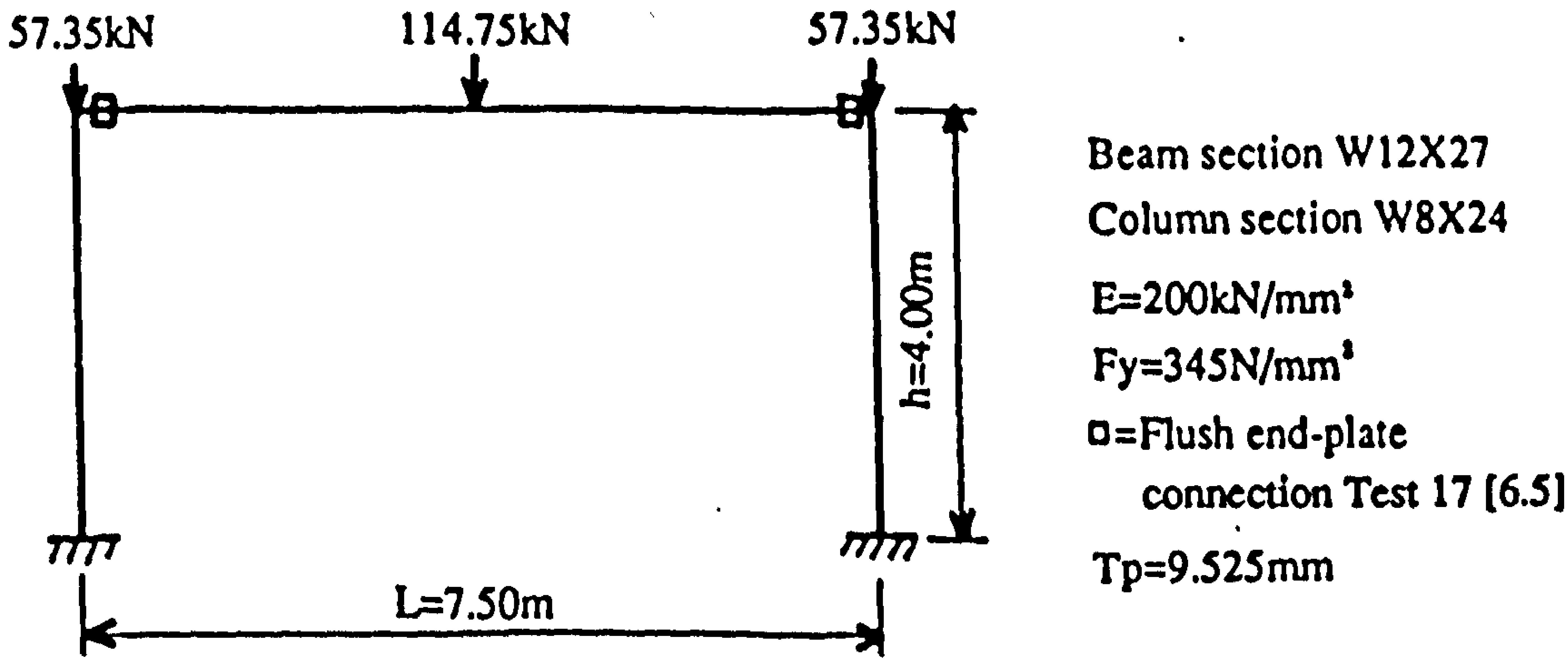
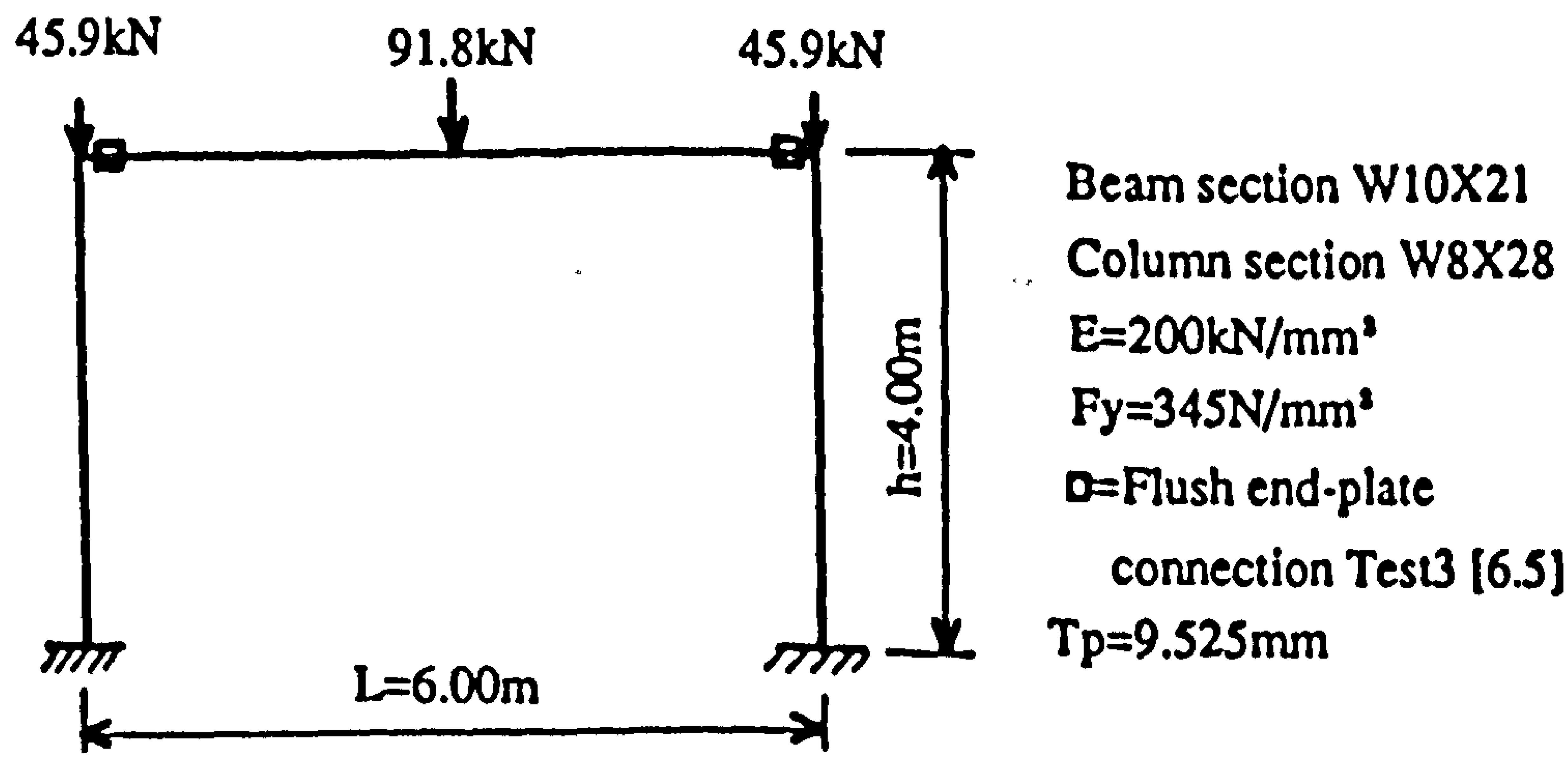


Figure 6.10 Effect of semi-rigid connections on sway deflection for Frame B, Type 1 loading.



a) Frame configuration S1



b) Frame configuration S2

FIG.6.11 Frame configuration and loading conditions for Frames S1 and S2

FIG.6-12a Effect of moment-rotation curves on the elastic analysis results of Frame S1.

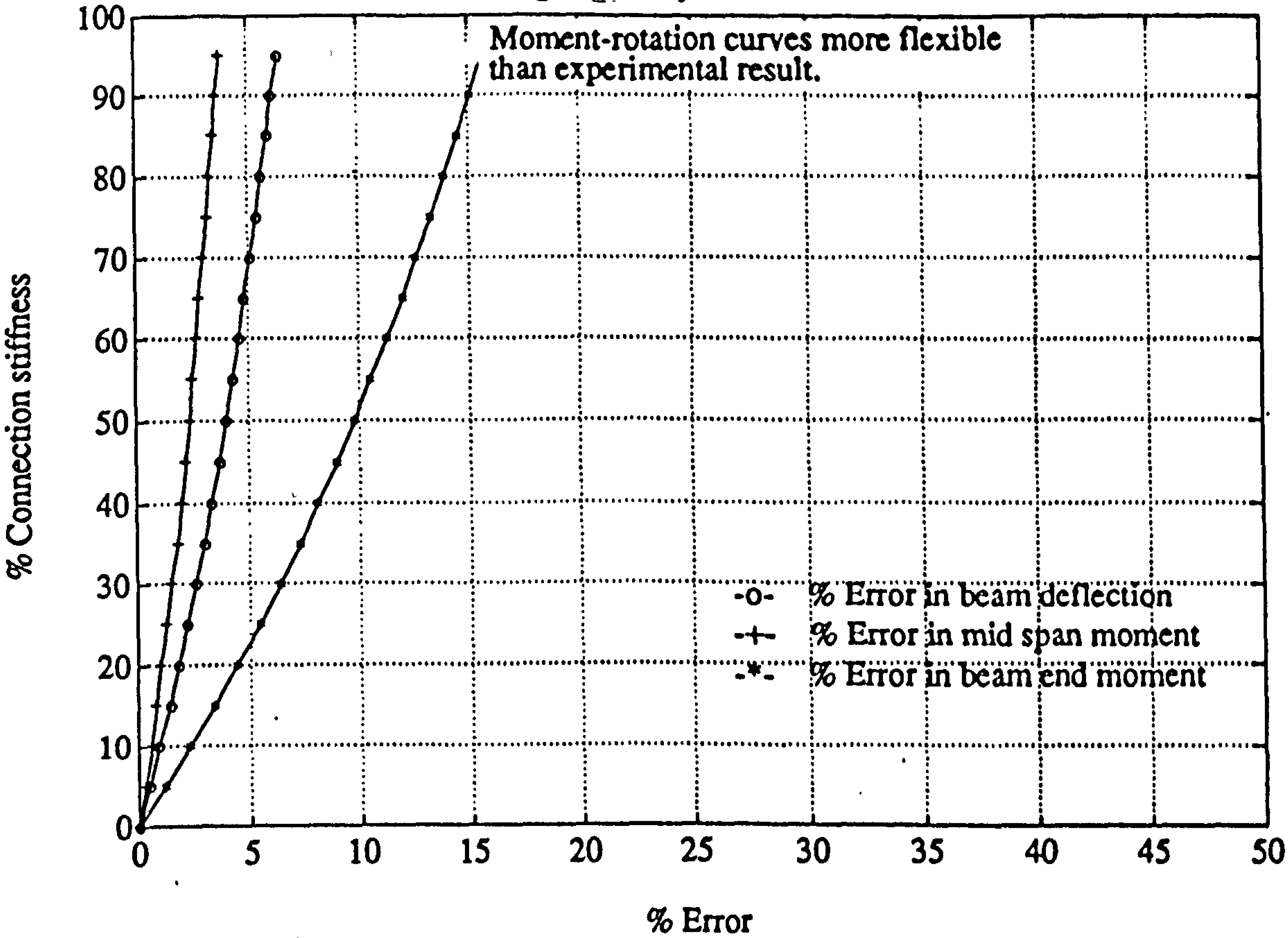


FIG.6-12b Effect of moment-rotation curves on the elastic analysis results of Frame S1.

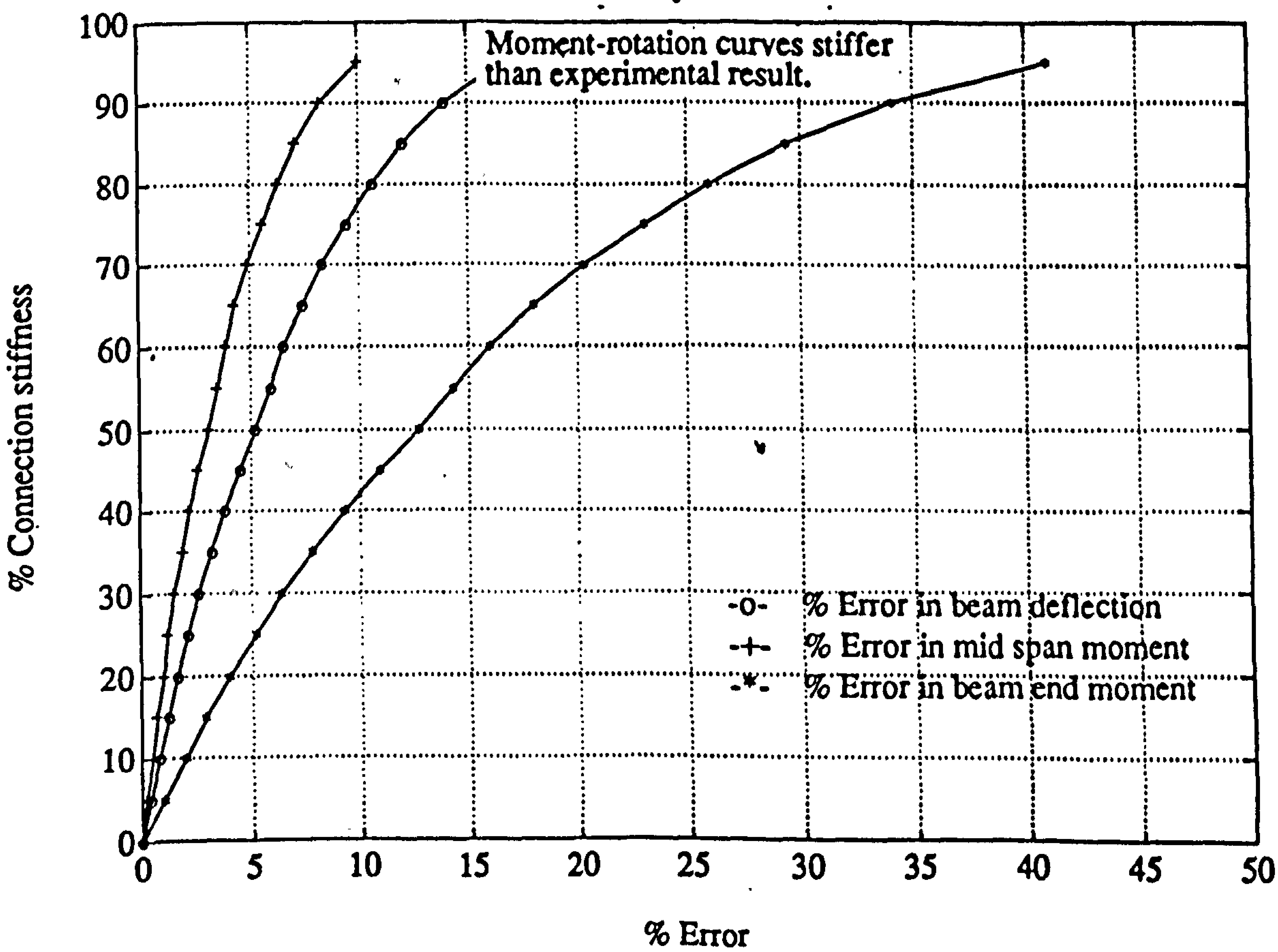


FIG.6-13a Effect of moment-rotation curves on the elastic analysis results of Frame S2.

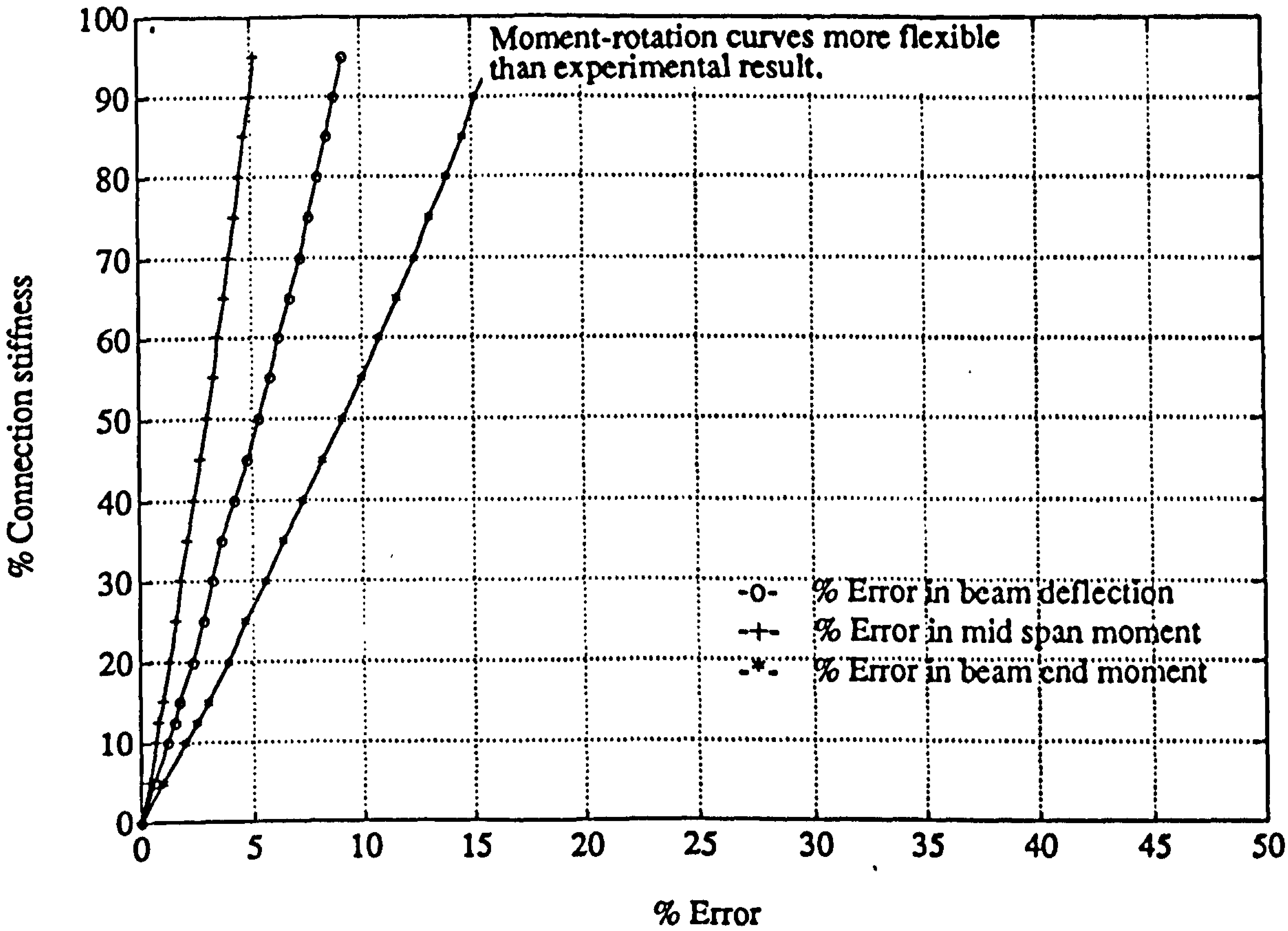


FIG.6-13b Effect of moment-rotation curves on the elastic analysis results of Frame S2.

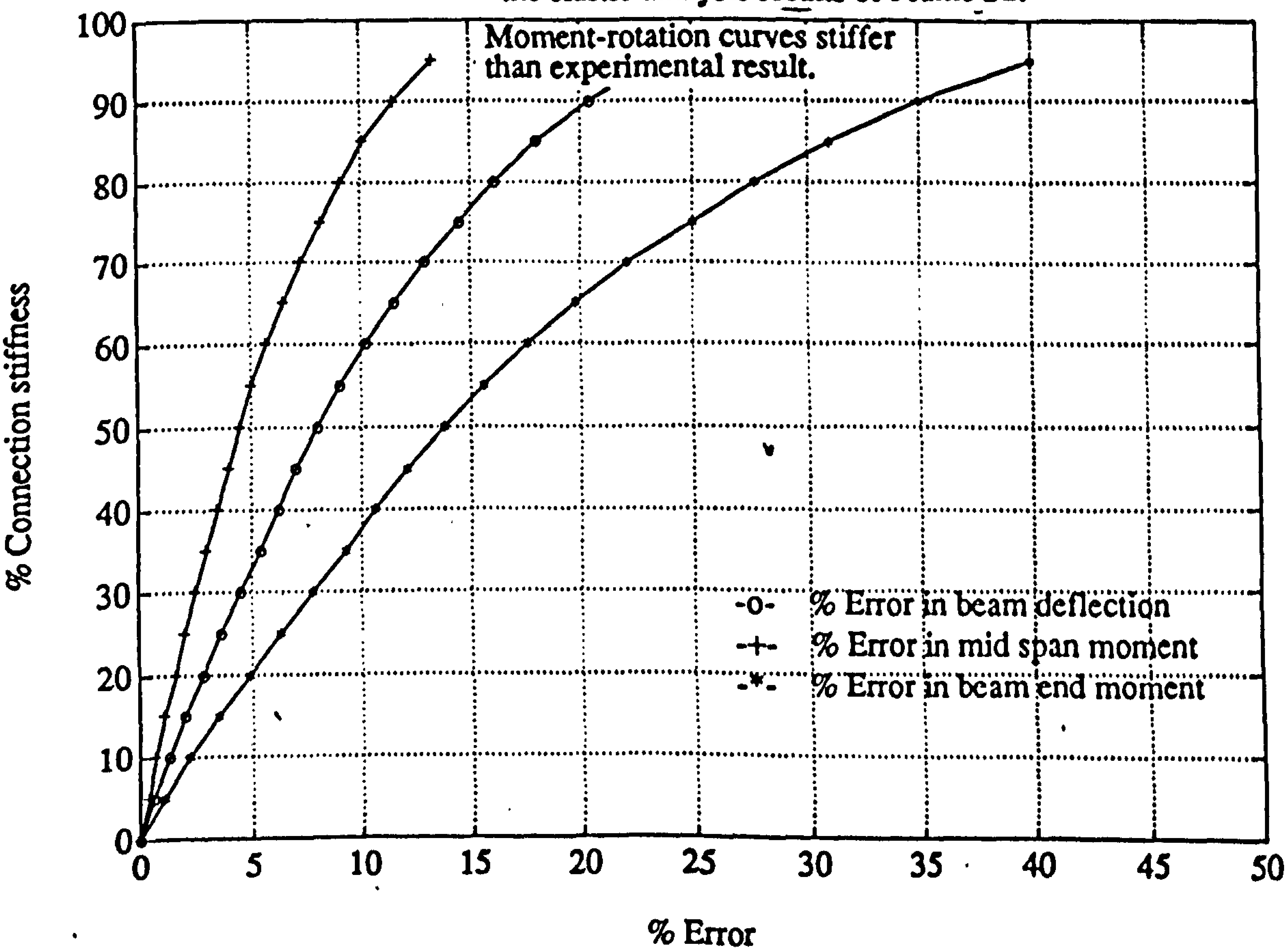


FIG.6-14a Effect of moment-rotation curves on the elastic analysis results of Frame A.

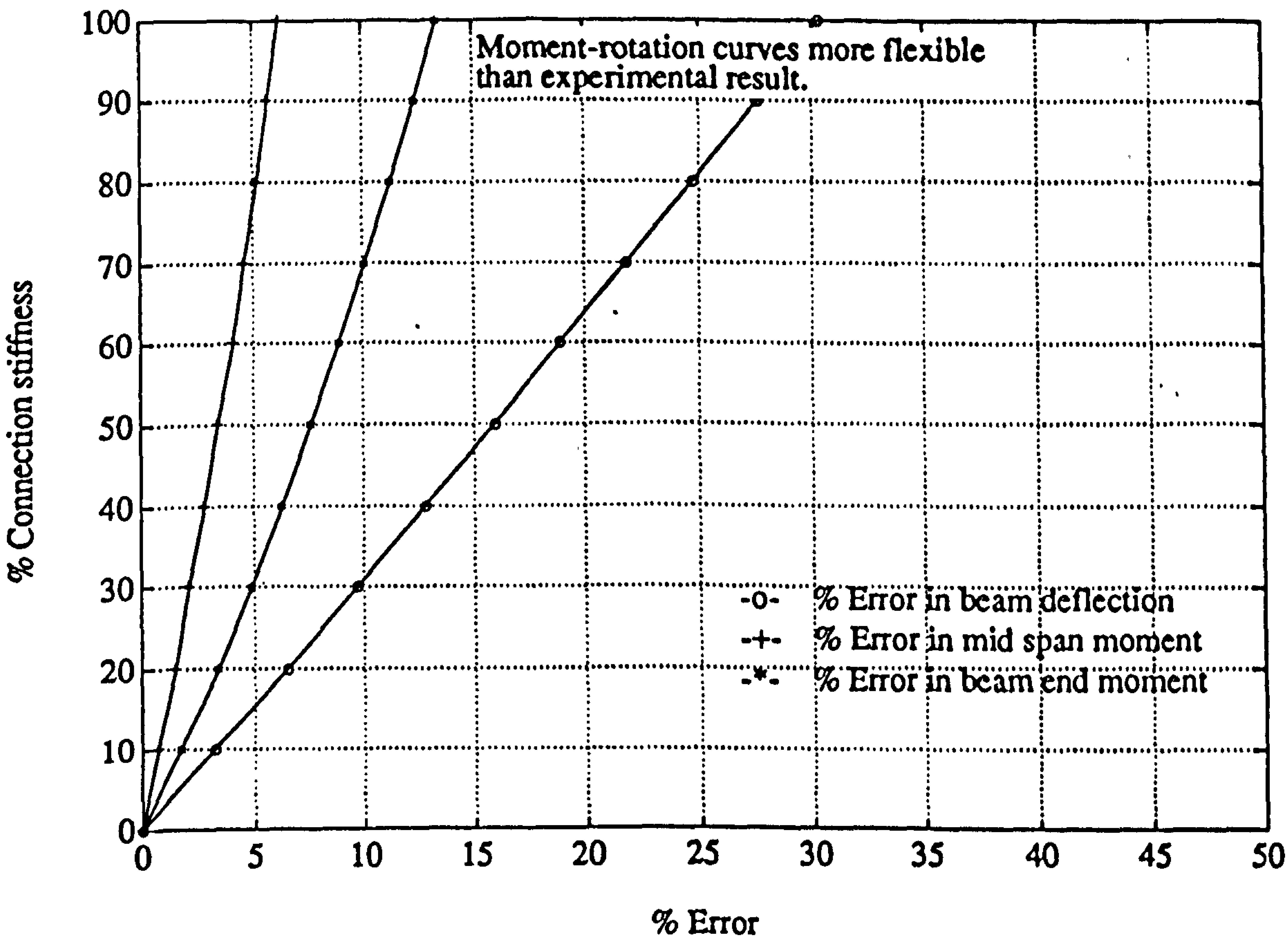


FIG.6-14b Effect of moment-rotation curves on the elastic analysis results of Frame A.

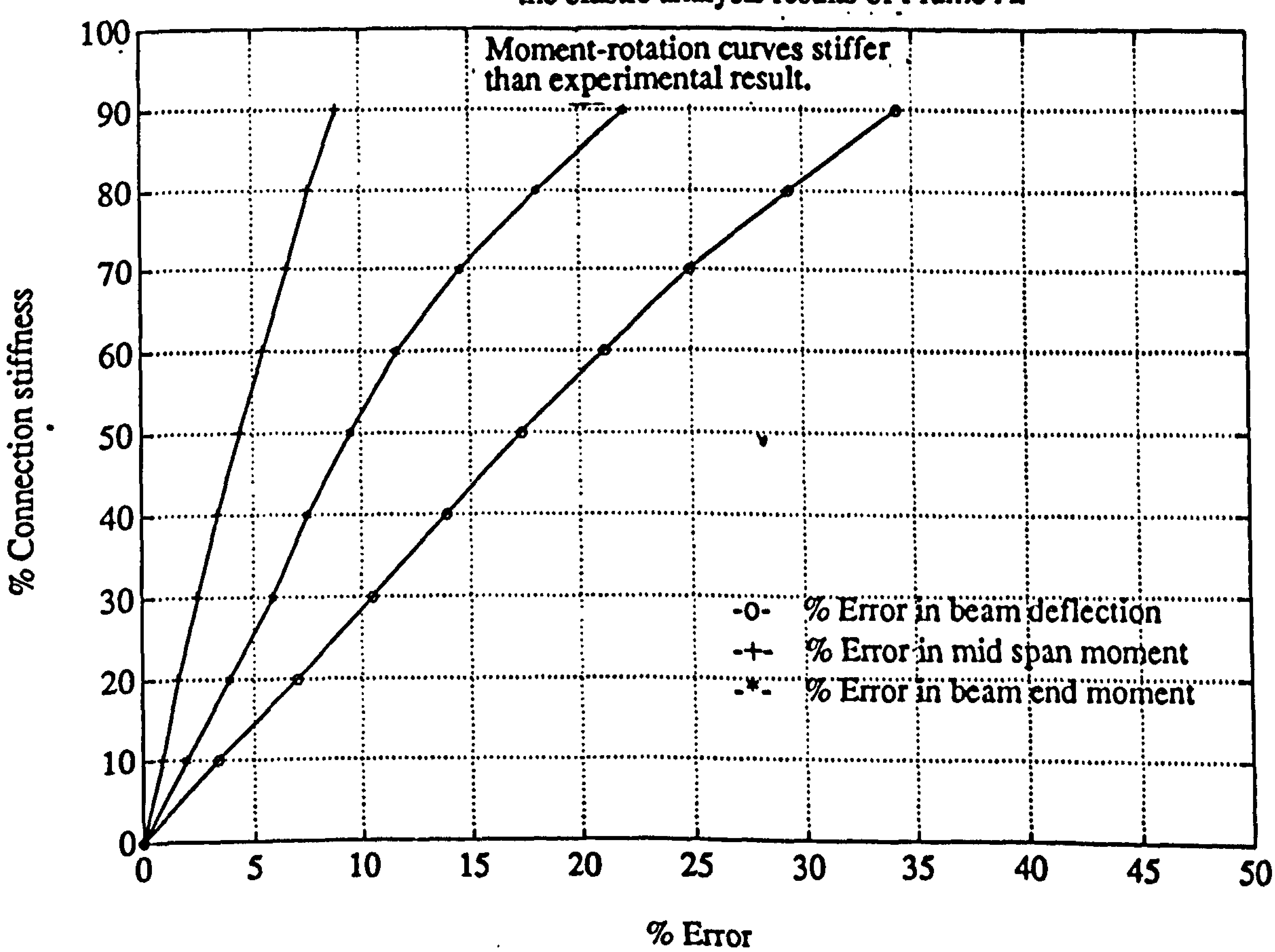
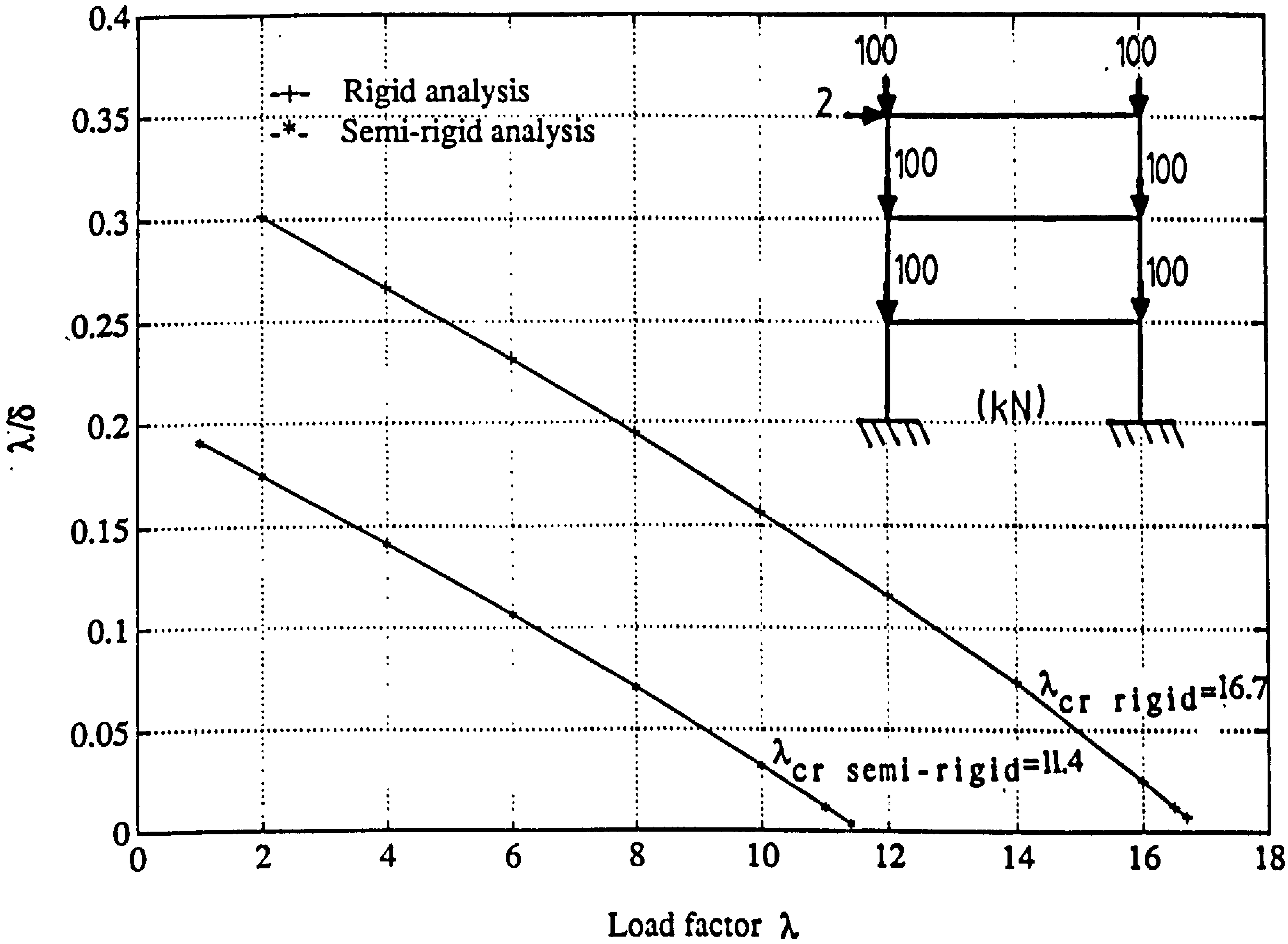


FIG.6-15 Elastic critical load for Frame A.



CHAPTER VII
SIMPLE SEMI-RIGID DESIGN OF STEEL FRAMES

7.1. Introduction

The design of a steel structure should at present be carried out in accordance with one of the methods given in clause 2.1.2 of BS5950 [7.1]. The design methods are:

i) Rigid design: In this method the connections between members are assumed to be capable of developing the strength and/or stiffness required by an analysis assuming full continuity. Such analysis may be made using either elastic or plastic methods.

ii) Simple design: The connections are assumed not to develop moments adversely affecting either the members or the structure as a whole. The structure is assumed to be pin jointed for analysis. A bracing system is necessary to provide resistance to horizontal loading.

iii) Semi-rigid design: Practical joints are capable of transmitting some moments and the method takes the partial fixity into account. Two approaches to semi-rigid design are given:

   a) Experimental determination of joint behaviour: some limited plasticity is permitted, but the ultimate tensile capacity of the fastener is not to be the failure criterion.

   b) Simplified empirical method: The inter-restraint of the beams and columns is taken as not more than 10% of the free moment in the beams. In addition, the code sets out other provisions which must be met when using the method (see clause 2.1.2.4).

In practice, structures are usually designed to either the rigid or simple methods of design. Because of connection flexibility, rigid analysis generally underestimates the beam moments, but overestimates the column moments. However, the simple framing method accounts for the effects of connection stiffness to some extent by assuming the beam reactions act at an eccentricity of 100 mm from the face of the column. This type of analysis overestimates the beam moments, but still underestimates the column moments.

In a real frame the moments carried by a beam subjected to a uniformly distributed load, ω, will lie somewhere between the value zero and $\omega L^2/12$ at the supports and $\omega L^2/24$ to $\omega L^2/8$ at the centre, depending on the relative stiffness of the connection, beam and column. The first, and even nowadays one of the few methods, that incorporated semi-rigid action was developed in the 1930's by the Steel Structures Research Committee [7.2], and was based on beam-line theory as described in Chapter 5. As stated earlier in this chapter, the recent British limit state design code [7.1], BS5950 clause 2.1.2.4, allows the designer to use semi-rigid action in one of two ways: firstly by basing the connection behaviour on experimental evidence when assessing moments and forces in the members; the alternative is to assume the beam-to-column connection provides end restraint equal to 10% of the free bending moment applied to the beam.

The aim of the study reported in this chapter is to check whether this allowance is appropriate for flush end-plate and extended end-plate semi-rigid connections at the ultimate limit state.

7.2 Experimental evidence

Whilst work on connections, members and subassemblages has provided much insight into the ways in which connection behaviour influences frame response, it is only by considering complete frames that the whole problem can be properly investigated.

Only a few large-scale tests on semi-rigidly connected frames have been tested, the recent BRE series being the most substantial. These tests contributed towards a collaborative effort by BRE, Hatfield Polytechnic and Sheffield University to study the influence of semi-rigid bolted connections on frame behaviour, under practical loading conditions. In total five frames were tested incorporating a wide spectrum of different end restraint, from extended end-plates to flange cleats. The first two tests employed frames with heavy columns, fairly stiff end-plate connections and were designed and tested by Hatfield Polytechnic, whilst the second pair used much lighter columns and cleated connections designed and tested by Sheffield University and finally the fifth frame by BRE.

7.2.1 Sheffield frame test results

Two full scale, flexibly connected, non-sway frames loaded up to failure were tested at the Building Research Establishment by Davison [7.3]. The sections used were 254x102x22UB for beams and 152x152x23UC for columns. The flange cleat type of connection was selected. This flexible type of connection is frequently used in steel frame connection because of the usefulness of the seating cleat during erection. In test frame 1, beams framed into the column flange and in test frame 2 beams framed into the column web. Both frames were three storeys high and two bays wide, as shown in Figures 7.1 and 7.2. The storey height was 3.60m and the beam width was 4.953m. Grade 43A steel and M16 grade 4.6 bolts were used throughout. The bolts were tightened to 106 Nm. The frames were tested in-plane and with no-sway, i.e. out-of-plane buckling and lateral displacement of the frame were prevented by bracing. The unfactored dead load per beam was 53 kN and the unfactored live load per beam was 26.5 kN. In both frames the loading of beam 5 was restricted to 53 kN, corresponding to unfactored dead load; the other beams were loaded to ultimate design values. In practice the loads could not be applied to the beams as uniformly distributed loads. Instead two point loads were applied to each beam, at the beam's quarter and three quarter points as shown in Figures 7.1 and 7.2. This configuration of loading represents the free moment diagram for a uniformly distributed load quite accurately by the use of just two point loads. The frames were designed in accordance with BS5950 [7.1] and BS449 [7.5]. The calculations were used to determine the level of beam loading which the chosen section would sustain at the ultimate condition and to estimate the design capacity of the columns. At the ultimate design load, the right and left beam end moments are shown in Table 7.1 and 7.2 for frames 1 and 2 respectively. The term $\omega L^2/8$ corresponds to the free bending moment in the centre segment of a simple supported beam of span L with a uniformly distributed load ω equal to 2Wl/L. As mentioned above, the free bending moment in the centre segment of the beam with the configuration of loading shown in Figure 7.1 corresponds exactly to the value $\omega L^2/8$.

As a result of these two three-storey, two-bay bare steel frames with flange cleat beam to column connections, the following findings are deduced:

i) The rotational stiffness of the simple connections influenced the distribution of moments around the frame.

ii) For frame 1, at a load level corresponding to the ultimate design load for the beam the connections attracted a moment of approximately 20% of the free bending moment in the centre segment of the beam except joints at beam ends number 5 which were subjected to smaller moments due to reduced load applied to their associated beam (see Table 7.1).

iii) For frame 2, in which beams were framed to column web, at the ultimate design load for the beam the connections sustained moments in the range 15 to 30% of the free bending moment dependent on the stiffness of the column and the presence of an equal load on an adjacent span (see Table 7.2).

iv) Moments transmitted by the connections were considerably greater than the values obtained from the beam end reaction acting at a nominal 100 mm eccentricity.

v) There is not much difference between beam end moments connected to external column and to connection to internal column. This is presumably because joint stiffness is the dominant factor, not stiffness of other members.

vi) Certainly as the flange cleat connections are far more flexible than the flush and the extended end-plate connections, one would expect with the latter connections an inter-restraint of the beams and columns greater than 20% of the free moment in the beams.

7.2.2 Hatfield frame test results.

Two further full scale frames with beams semi-rigidly connected to the column flanges, were loaded and tested at BRE by Prescott [7.4]. The sections were 254x146x43UB for beams and 203x203x71UC for the columns. The frames were three-storeys high and two-bays wide. The storey height was 3.600m; the bay width was 3.810m for span A and 6.096m for span B, as shown in Figure 7.3. Flush end-plate connections were used for span A and the stiffened extended end-plate connections were used for span B. Grade 43A steel and M20 grade 8.8 bolts were used throughout the frames. In frame 1 the bolts were hand tightened, corresponding to 25 kN preload, and in frame 2 the bolts were preloaded to 100 kN. Two point loads were applied to each beam at its quarter and three quarter as in frames tested by Davison [7.3]. The following design loads were considered:

Dead load	
Roof beam and floor level	2.12 kN/m
Live load	
Roof beams	3.0 kN/m
Floor beams	10.2 kN/m

Both frames were restrained transversely at the following positions: (i) for beams, at the quarter and three quarter point and mid span (ii) for columns, at each floor level and mid storey height. In addition for no-sway tests, the frame was restrained in its own plane at each floor level by the in-plane sway bracing bars at the left end beam connections of span B. The frames were tested at different stages of loading; preliminary testing (low loads), working load (D.L. + L.L.) and design load (1.4D.L. + 1.6L.L.) with sway and no sway modes. Also sufficient loads to cause failure of the frame were applied with the frame restrained from swaying for the test to failure.

Frame test 1 results at the ultimate design load are not available in reference [7.4]. In frame test 2, at ultimate design load, the beam end moments are nearly the same whether or not sway was permitted. For the frame prevented from sway the experimental beam end moments are given in Table 7.3 as well as the theoretical rigid beam end moments. On the basis of this test results, the present author concluded that:

i) In all cases the connection at each end of the beam attracted a very high proportion of the free bending moment, at a load level corresponding to the ultimate design load for beam. These distributions of bending moments depend upon the relative stiffness of members and the joints as well as on the relative magnitude of loadings.

ii) As illustrated by the rigid analysis results, one would expect the beam moments at the ends adjoining the external column of span A to be greater when the frame is permitted to sway, as opposed to being braced. Similarly for span B, the greater end moments would be expected at the central column. This was found to be so in the majority of cases [7.4].

iii) In span A beam 4 the left end attracted 105% of the free bending moment with 32% of M_f on the other end. This results from the greater stiffness of members and the joint at the left end of the span and the effect of much thicker plate used.

iv) The extended end-plate connections attracted a minimum moment of approximately 50 to 60% of the free bending moment, as shown by the left end of span B. This results from thick column flanges and the effect of thick end-plate connections.

As a result of these above experimental investigations, a possible modification to the BS5950 method would be to take more account of this restraint where connections are capable of sustaining higher moments, as it was clearly demonstrated for the above full scale frames.

7.3 Design method to include the benefit of semi-rigid action

As earlier mentioned, there are two alternative approaches to elastic design of braced frames with semi-rigid connections. Firstly, approximate allowance may be made for the stiffness of the connections by a limited redistribution of moment. In the second approach, the stiffness is represented by M-Φ curves which are included in the analysis of the frame. The components are then sized on the basis of the resulting moments and forces. The approximate design by limited redistribution of moment is considered below.

7.3.1 Beam design

The allowance made for semi-rigid design in BS5950 clause 2.1.2.5, termed the "10% rule", allows 10% of the free bending moment, M_f, to be carried by the connection, resulting in the beam being designed for 90% of M_f. Therefore the principal benefits of semi-rigid action appears to be in the reduction of beam span moments, deflection, and hence beam section sizes.

7.3.2 Column design

The columns must be designed to resist the algebraic sum of the restraint moments from the beams at the same level on each side of the column, in addition to moments due to eccentricity of connections described in BS5950 clause 4.7.7. This traditional British approach, first advanced in the 1930's, is to assume the moment to be equal to a value obtained by multiplying the beam end reaction by a notional eccentricity of 100mm from column face. The method adopted here is to assume the columns axially loaded with an end-moment due to semi-rigid connection as shown in Figure 7.4. The nominal moments due to the 100mm eccentricities were not considered. The reasons for neglecting the eccentricity moments are as follows:

i) to give wider application to the study as this is not included in European practice (i.e. not in EC3);

ii) the effect of including eccentricity moments would be to give bigger column sections and consequently more end restraint. Therefore, the effect of neglect of eccentricity moments is conservative for beam design.

Transfer of beam end moments to the adjacent columns in accordance with BS5950 cl. 4.7.7 is shown in Figure 7.4. It is clear from this figure that the end moments applied at any floor level should be divided between the column lengths above and below that level in proportion to the stiffness, I/L, of each length, except that when the ratio of stiffness does not exceed 1.5 the moment may be divided equally.

For the frames selected for analysis the storey height was the same. To avoid iteration in design, at any floor level a proportion of one third of the beam end moment was assumed to be taken by the column above and a two-third of the beam end moment is taken by the lower column at that level.

In BS5950 cl. 2.1.2.4, the benefit of the beam end restraint is not accounted for in reduction of the column's effective length. In any case, it is minor axis buckling that will control.

7.3.3 Connection design

The beam-to-column connections were designed to transmit the appropriate restraint moment, in addition to the end reactions assuming the beams were simply supported. Two types of connections were used: 1) the flush end-plate and 2) the end-plate connections extended on the tension side only. Columns were assumed to be unstiffened. The flush end-plate connections were designed according to the design methods presented in BCSA's "Manual on connections" described by Pask [7.6]. The extended end-plate connections were designed according to Horne and Morris [7.7]. With regard to the distortion caused by welding it is advisable to use a minimum end-plate thickness of 8mm for flush end-plate and 12.5 mm for extended end-plate connections.

7.4 Selected frames data and analysis results

7.4.1 Frame data

In this study, it was assumed initially that the beam-to-column connection provided an end restraint equal to 10% of the free bending moment applied to the beam as described in BS5950 cl. 2.1.2.4. A revised design was then made assuming end moments equal to 20% of the free bending moment. In total 56 frames were studied: 6 single storey-one bay frames, 6 two storey-one bay and 2 of two storey-two bay frames, each being designed by the two methods described above for the two types of connection (flush end-plate and extended end-plate). The rectangular frames analysed are shown in Figures 7.5, 7.6 and 7.7, together with the loads considered in the analysis. The frames were considered to represent commercial structures. The spacing of the main frames in the structure was taken 4.00m. A suitable storey height for such building was assessed to be 4.00m. The span of the frames ranged from 5.00m to 10.00m, except that for the two storey-two bay frames where the span was taken 5.00m. Grade 43A steel and M20 grade 8.8 bolts were used throughout the frames. The bolts were assumed to be preloaded to 50kN. The following realistic design loads were used:

Dead load	
Flat roof steelwork	16.0 kN/m
Floor steelwork office use	46.0 kN/m
Live load	
Flat roof	6.0 kN/m
Floor	14.0 kN/m

The load factors, γ_f, are 1.4 for dead load and 1.6 for live load. Therefore for roof the ultimate load "$\omega 1$" is 32 kN/m and for floor "$\omega 2$" is 86.8 kN/m. In the analysis, the uniformly distributed load is taken as point loads, spaced 1.0m apart. In one series of the two storey-two bay frames, the ultimate design load was applied on the right span beams and only the factored dead load on the other beam spans (see Figure 7.7a). In the other series of such frames the loading conditions at the floor level were altered as shown in Figure 7.7b.

This presence of unequal loading on an adjacent span was applied to see its effect on the beam end moments.

The notation used in this chapter for frame identification number is as follows:

The first letter F represents "frame" followed either by the letter F or E, where F indicated that flush end-plate connections were used throughout the frame or E for extended end-plate connections. The number following the letter F or E refers to the identification number of the frame which depends on the span length of the beam, number of storeys and number of bays (i.e. two frames with the same frame geometry and loading conditions with either flush or extended end-plate connections have the same identification number). The letters A and B indicate respectively that a frame is designed according to the 10% rule and to the 20% rule.

7.4.2. Moment-rotation relationship used for frame analysis

For consitency of analysis it is preferred to use the same model to predict M-Φ behaviour of the connection but unfortunately it is not possible. Therefore, it was decided to use the modified Frye and Morris [7.8] prediction equation (equ. (4.11)) for extended end-plate connections and the proposed equation by the author for the flush end-plate connections (equ. (3.24)). The moment capacity of the connection was calculated using the method proposed by Horne and Morris [7.7] for extended end-plate connections based on the smallest values given by the moment capacity of the chosen end-plate thickness, column flange, column web in compression and column web in shear. For the flush end-plate connections the limiting moment capacity of the connection is given by the factor C_1 in the M-Φ prediction equation (equ. (3.24b)).

7.4.3 Analysis results

The results of analysis and calculations are summarised in Tables 7.4 and 7.5 for single storey-one bay frames and two storey-one bay frames with flush end-plate connections. Tables 7.6 and 7.7 show the results for single storey-one bay frames and two storey-one bay frames with extended end-plate connections. Table 7.8 represents the results for two storey-two bay frames. On each table, results based on 10% and 20% design rule are given. The following notations are used:

T_p = end-plate thickness,

T_{fc} = column flange thickness,

D_b = beam depth,

$I_b \cdot h/\Sigma(I_c \cdot L)$ = stifness ratio, i.e. ratio of the beam stiffness to the summation of column stiffnesses meeting at that joint,

* = one bolt row in the tension zone of the connection, and

** = two bolt rows in the tension zone of the connection.

For all the extended end-plate connections four bolts were used in the tension zone of the end-plate. The second bolt row is taken into account in the M-Φ relationship for the flush end-plate connection.

The beam end moments are given in terms of percentage of the free bending moment of a simply supported beam carrying the same loading. The only parameters used here as variables in designing the connections were: i) end-plate thickness, ii) beam depth; iii) column flange thickness and iv) contribution of a second bolt row below the beam tension flange. The vertical pitch distance, horizontal gauge distance, yield stress, bolt's size, bolt's preload force and proof load were kept constant for all the connections.

In general for frames designed for 10% rule, at a load level corresponding to the ultimate design load for the beam, the connections sustained moments in the range of 12 to 20% of the free bending moment dependent on the stiffness of the columns and connections.

It will be noted from comparisons of Tables 7.4, 7.5, 7.6 and 7.7 and results from Table 7.8 that the stiffness ratios for frames with extended end-plate connections are greater than those with flush end-plate connections. This is due to stronger column used in frames employing flush end-plate connections, the column sections being decided by the resistance required from the column flange in the tension region of the connection.

For frames designed for 20% rule, at a load level corresponding to the ultimate design load for the beam, the frame analysis results demonstrate that the beam end moments were of 20 to 30% of the free bending moment. The results are shown in Tables 7.4 to 7.8. As mentioned in the above 10% rule, the frames with extended end-plate connections have a higher stiffness ratio in comparison with flush end-plate connected frames. Contrary to expectations for the same frames, (i.e. same beam span, section and loading), some frames with flush end-plate connections attract more moments than the ones with extended end-plate connections. This is due to the thicker column section used, based on the Pask design method [7.6] and to the limiting moment capacity.

For the two storey-two bay frames shown in Table 7.8, connections attracted the same amount of moment as for frames with two storey-one bay frames for the span with ultimate design load (such as FF7A with FF21A, FF7B with FF21B, FE7A with FE21A and FE7B with FE21B). Connections in spans with only factored dead load attracted a lower moment.

Figures 7.8 and 7.9 summarise the analysis results of the beam end moments for connections designed based on the 10% rule and 20% rule respectively. Analysis results for the two storey-two bay frames are not included.

7.5 Conclusions

Existing proposals for semi-rigid design using limited redistribution of moment do not show significant economy over "simple" design. This is primarily a result of the low degree of redistribution that is permitted at present in design codes. Experimental results reported by other authors on full scale three storey-two bay bare steel frames with semi-rigid beam-to-column connections have shown the following results:

i) At the ultimate design load for the beam the unstiffened flange cleat connections attracted a moment of approximately 20% of the free bending moment in the centre segment of the beam.

ii) The stiffened flush end-plate connections sustained a minimum moment of approximately 30 to 40% of the free bending moment dependent on the stiffness of the column.

iii) The stiffened extended end-plate connections provided a minimum end restraint of approximately 50 to 60% of the free bending moment.

In all frames studied, the distribution of beam end moments depends on the relative stiffness of members, the joints and the presence of an equal load on an adjacent span. Frames designed with 10% rule result in a weaker column section; in these cases $I_b.h/\Sigma(I_c.L)$ is high, and therefore the joint attracts a low value of moment which was between 12 to 20% of the free bending moment. Frames designed with 20% rule end up by having a strong column section, i.e. low value of $I_b.h/\Sigma(I_c.L)$, and therefore the joints attract higher moments than the frames designed based on 10% rule. The connections attracted a moment of 20 to 30% of the free bending

10% rule. The connections attracted a moment of 20 to 30% of the free bending moment. This is represented in a schematic way in Figure 7.10.

The analysis results found by the present author proved less restraint than the experimental tests. This is due to a number of reasons: i) the column flanges are very stiff in comparison to the one used in the theoretical studies; ii) the end-plates are much thicker than the one used in the present investigation; and iii) the limiting moment capacity of the connection is very low in comparison to the experimental results of the Hatfield frames. The theoretical methods seem to underestimate real ultimate moment in connections.

In general, at a load level corresponding to the ultimate design load for the beam, the connections sustained a moment greater than the one which it was designed for. It is recommended that the BS5950 simplified method should permit 20% end restraint, which would improve significantly the attractiveness of the method.

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Beam ref no.	Left end (kNm)	Left end ($x\frac{\omega L^2}{8}$)	Right end (kNm)	Right end ($x\frac{\omega L^2}{8}$)
2	-15.51	21.5%	-17.68	24.5%
3	-16.57	22.9%	-14.75	20.4%
4	-14.93	20.7%	-14.90	20.6%
5	-10.29	15.1%	-13.56	18.8%
6	-14.89	20.6%	-16.50	22.8%

Table 7.1 Distribution of beam end moments for Sheffield Frame 1

Beam ref no.	Left end (kNm)	Left end ($x\frac{\omega L^2}{8}$)	Right end (kNm)	Right end ($x\frac{\omega L^2}{8}$)
1	-12.33	17.0%	-21.67	30.0%
2	-14.13	19.6%	-16.64	23.0%
3	-15.51	21.5%	-19.74	27.3%
4	-21.04	29.0%	-12.58	17.4%
5	-12.76	17.7%	-10.35	14.3%
6	-22.78	31.6%	-13.53	18.7%

Table 7.2 Distribution of beam end moments for Sheffield Frame 2

Beam Ref No.	Left end Rigid theory (kNm)	Left end Experimental (kNm)	Left end Experimental ($x\frac{\omega l^2}{8}$)	Right end Rigid theory (kNm)	Right end Experimental (kNm)	Right end Experimental ($\frac{\omega l^2}{8}$)
1	-89.5	-85	(53%)	-133.2	-113	(70%)
2 Span B	-140.6	-135	(64%)	-161.1	-135	(64%)
3	-133.6	-122	(57%)	-161.1	-134	(63%)
4	-68.8	-66	(105%)	-32.2	-20	(32%)
5 Span A	-71.4	-49	(59%)	-56.5	-31	(37%)
6	-82.6	-66	(80%)	-47.6	-41	(49%)

Table 7.3 Distribution of beam end moments for Hatfield Frame 2
(See section 7.2.2 for details of conection types)

Table 7.4 Analysis results for one storey - one bay frames with flush end-plate connections

Frame identification number	Span (m)	Beam section "UB"	Column section "UC"	Connection parameters			$\frac{I_b h}{\Sigma I_c L}$	Beam end moment x ($\omega L^2/8$)
				T_p (mm)	T_{fc} (mm)	D_b (mm)		
FF1A 10% rule	5.00	305x102x25**	152x152x30	8.0	9.4	304.8	2.02	16.4%
FF1B 20% rule		254x102x25*	203x203x52*	12.5	12.5	257.0	0.52	25.8%
FF2A 10% rule	6.00	305x102x33**	152x152x30	10.0	9.4	312.7	2.49	14.5%
FF2B 20% rule		305x102x33**	203x203x52	12.5	12.5	312.7	0.82	25.0%
FF3A 10% rule	7.00	356x127x39**	125x152x30	10.0	9.4	352.8	3.32	12.15%
FF3B 20% rule		305x127x42**	203x203x60	15.0	14.2	306.6	0.764	25.4%
FF4A 10% rule	8.00	305x165x54**	152x152x37	12.5	11.5	310.9	2.635	13.14%
FF4B 20% rule		356x171x45**	203x203x60	15.0	14.2	352.0	0.99	22.7%
FF5A 10% rule	9.00	457x152x52**	152x152x37	12.5	11.5	449.8	4.26	14.4%
FF5B 20% rule		356x171x57**	203x203x71	20.0	17.3	358.6	0.93	29.5%
FF6A 10% rule	10.00	406x178x67**	203x203x52	12.5	12.5	409.4	1.85	12.4%
FF6B 20% rule		406x178x60**	203x203x71	20.0	17.3	406.4	1.12	27.5%

Table 7.5 Analysis results for two storey-one bay frames with flush end-plate connections

Frame identification number	Span (m)	Beam section "UB"	Column section "UC"	Connection parameters T_p (mm)	T_{fc} (mm)	D_b (mm)	$\frac{I_b h}{\Sigma I_c L}$	Beam end moment x ($\omega L^2/8$)
FF7A 10% rule	5.00	305x102x25**	152x152x30	8.0	9.4	304.8	2.02	16.7%
		356x171x51**	152x152x37	12.5	11.5	355.6	2.87	13.5%
FF7B 20% rule	5.00	254x102x25*	203x203x52	12.5	12.5	257.0	0.52	26.4%
		305x165x54**	203x203x71	20.0	17.3	310.9	0.72	28.3%
FF8A 10% rule	6.00	305x102x33**	153x152x30	10.0	9.4	312.7	2.49	14.7%
		457x152x60**	153x152x37	12.5	11.5	454.7	4.29	11.6%
FF8B 20% rule	6.00	305x102x33**	203x203x52	12.5	12.5	312.7	0.82	25.4%
		406x178x60**	203x203x71	20.0	17.3	406.4	1.11	26.8%
FF9A 10% rule	7.00	356x127x39**	152x152x30	10.0	9.4	352.8	3.32	12.2%
		457x191x82**	203x203x52	12.5	12.5	460.2	3.21	10.2%
FF9B 20% rule	7.00	305x127x42**	203x203x60	15.0	14.2	306.6	0.76	25.8%
		457x152x74**	203x203x71	20.0	17.3	461.3	1.35	23.2%

Table 7.5 continued

Frame identification number	Span (m)	Beam section "UB"	Column section "UC"	Connection parameters T_p (mm)	T_{fc} (mm)	D_b (mm)	$\frac{I_b h}{\Sigma I_c L}$	Beam end moment x ($\omega L^2/8$)
FF10A 10% rule	8.00	305x165x54**	152x152x37	12.5	11.5	310.9	2.63	13.3%
		533x210x92**	203x203x52	12.5	12.5	533.1	3.70	14.2%
FF10B 20% rule	8.00	356x171x45**	203x203x60	15.0	14.2	352.0	0.99	23.0%
		533x210x82**	203x203x71	20.0	17.3	528.8	1.73	20.5%
FF11A 10% rule	9.00	457x152x52**	152x152x37	12.5	11.5	449.8	4.26	14.4%
		610x229x101**	203x203x60	15.0	14.2	602.2	4.05	11.1%
FF11B 20% rule	9.00	356x171x57**	203x203x71	20.0	17.3	358.6	0.93	30.0%
		533x210x109***	203x203x71	20.0	17.3	539.5	1.94	16.7%
FF12A 10% rule	10.00	406x178x67**	203x203x52	12.5	12.5	409.4	1.85	12.4%
		610x229x125**	203x203x60	15.0	14.2	611.9	3.47	9.94%
FF12B 20% rule	10.00	406x178x60**	203x203x71	20.0	17.3	406.4	1.12	27.9%
		610x229x113***	203x203x86	20.0	20.5	607.3	2.04	18.74%

Table 7.6 Analysis results for one storey-on bay frames with extended end-plate connections

Frame identification number	Span (m)	Beam section "UB"	Column section "UC"	Connection parameters			$\frac{I_b h}{\Sigma I_c L}$	Beam end moment x ($\omega L^2/8$)
				T_p (mm)	T_{fc} (mm)	D_b (mm)		
FE1A 10% rule	5.00	305x102x25	152x152x23	12.5	6.8	304.8	2.79	20.4%
FE1B 20% rule		254x102x25	152x152x23	12.5	6.8	257.0	2.16	20.8%
FE2A 10% rule	6.00	305x102x33	152x153x23	12.5	6.8	312.7	3.43	17.8%
FE2B 20% rule		305x102x33	152x152x23	12.5	6.8	312.7	3.43	17.8%
FE3A 10% rule	7.00	356x127x39	152x152x23	12.5	6.8	352.8	4.58	15.4%
FE3B 20% rule		305x127x42	152x152x30	15.0	9.4	306.6	2.67	22.3%
FE4A 10% rule	8.00	305x165x54	152x152x23	12.5	6.8	310.9	4.64	11.7%
FE4B 20% rule		356x171x45	152x152x37	15.0	11.5	352.0	2.72	23.7%
FE5A 10% rule	9.00	457x152x52	152x152x30	12.5	9.4	449.8	5.44	15.1%
FE5B 20% rule		356x171x57	203x203x46	15.0	11.0	358.6	1.57	23.2%
FE6A 10% rule	10.00	406x178x67	152x152x37	12.5	11.5	409.4	4.38	15.0%
FE6B 20% rule		406x178x60	203x203x46	15.0	11.0	406.4	1.89	21.25%

Table 7.7 Analysis results for two storey-one bay frames with extended end-plate connections

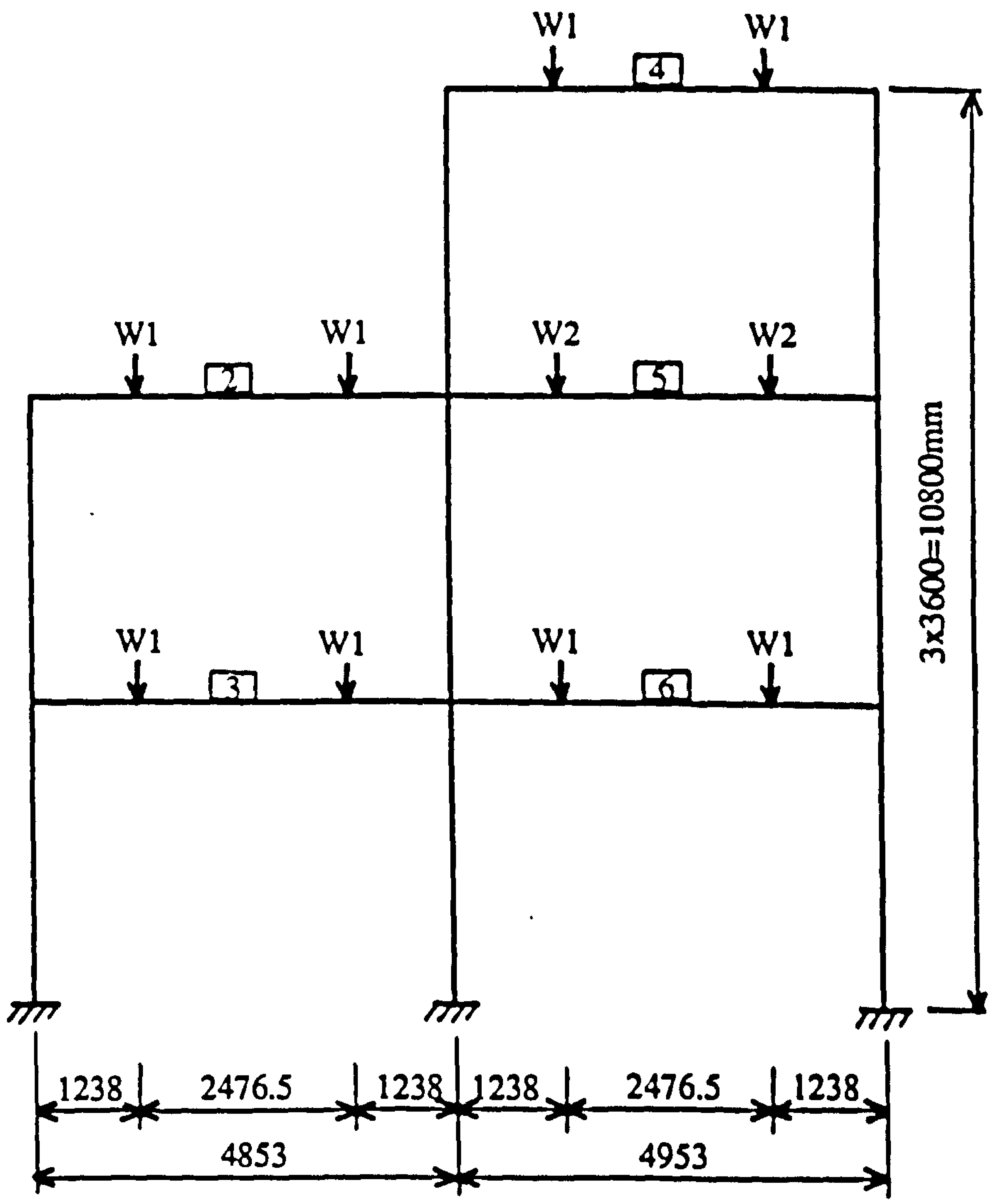
Frame identification number	Span (m)	Beam section "UB"	Column section "UC"	Connection parameters			$\frac{I_b h}{\Sigma I_c L}$	Beam end moment x ($\omega L^2/8$)
				T_p (mm)	T_{fc} (mm)	D_b (mm)		
FE7A 10% rule	5.00	305x102x25	152x152x23	12.5	6.8	304.8	2.79	23.8%
		356x171x51	152x152x30	12.5	9.4	355.6	3.79	16.6%
FE7B 20% rule	5.00	254x102x25	152x152x23	12.5	6.8	257.0	2.16	22.0%
		305x165x54	203x203x46	15.0	11.0	310.9	1.61	22.1%
FE8A 10% rule	6.00	305x102x33	152x152x23	12.5	6.8	312.7	3.43	20.1%
		457x152x60	152x152x37	12.5	11.5	454.7	4.88	15.4%
FE8B 20% rule	6.00	305x102x33	152x152x23	12.5	6.8	312.7	3.43	19.9%
		406x178x60	203x203x46	15.0	11.0	406.4	2.46	21.7%
FE9A 10% rule	7.00	356x127x39	152x152x23	12.5	6.8	352.8	4.58	16.9%
		457x191x82	203x203x46	12.5	11.0	460.2	3.64	13.2%
FE9B 20% rule	7.00	305x127x42	152x152x30	15.0	9.4	306.6	2.67	24.7%
		457x152x74	203x203x46	20.0	11.0	461.3	2.94	20.7%

Table 7.7 continued

Frame identification number	Span (m)	Beam section "UB"	Column section "UC"	Connection parameters			$\frac{I_b h}{\Sigma I_c L}$	Beam end moment x ($\omega L^2/8$)
				T_p (mm)	T_{fc} (mm)	D_b (mm)		
FE10A 10% rule	8.00	305x165x54	152x152x23	12.5	6.8	310.9	4.64	11.7%
		533x210x92	203x203x46	12.5	11.0	533.1	4.76	13.0%
FE10B 20% rule	8.00	356x171x45	152x152x37	15.0	11.5	352.0	2.72	25.4%
		533x210x82	203x203x60	20.0	14.2	528.3	2.86	24.2%
FE11A 10% rule	9.00	457x152x152	152x153x30	12.5	9.4	449.8	5.44	17.7%
		610x229x101	203x203x46	12.5	11.0	602.2	5.34	12.5%
FE11B 20% rule	9.00	356x171x57	203x203x46	15.0	11.0	358.6	1.57	23.2%
		533x210x109	203x203x71	20.0	17.3	539.5	2.43	22.2%
FE12A 10% rule	10.00	406x178x67	152x152x37	12.5	11.5	409.4	4.38	15.0%
		610x229x125	203x203x60	12.5	14.2	611.9	4.75	16.4%
FE12B 20% rule	10.00	406x178x60	203x203x46	15.0	11.0	406.4	1.89	21.25%
		610x229x113	203x203x86	20.0	20.5	607.3	2.49	22.6%

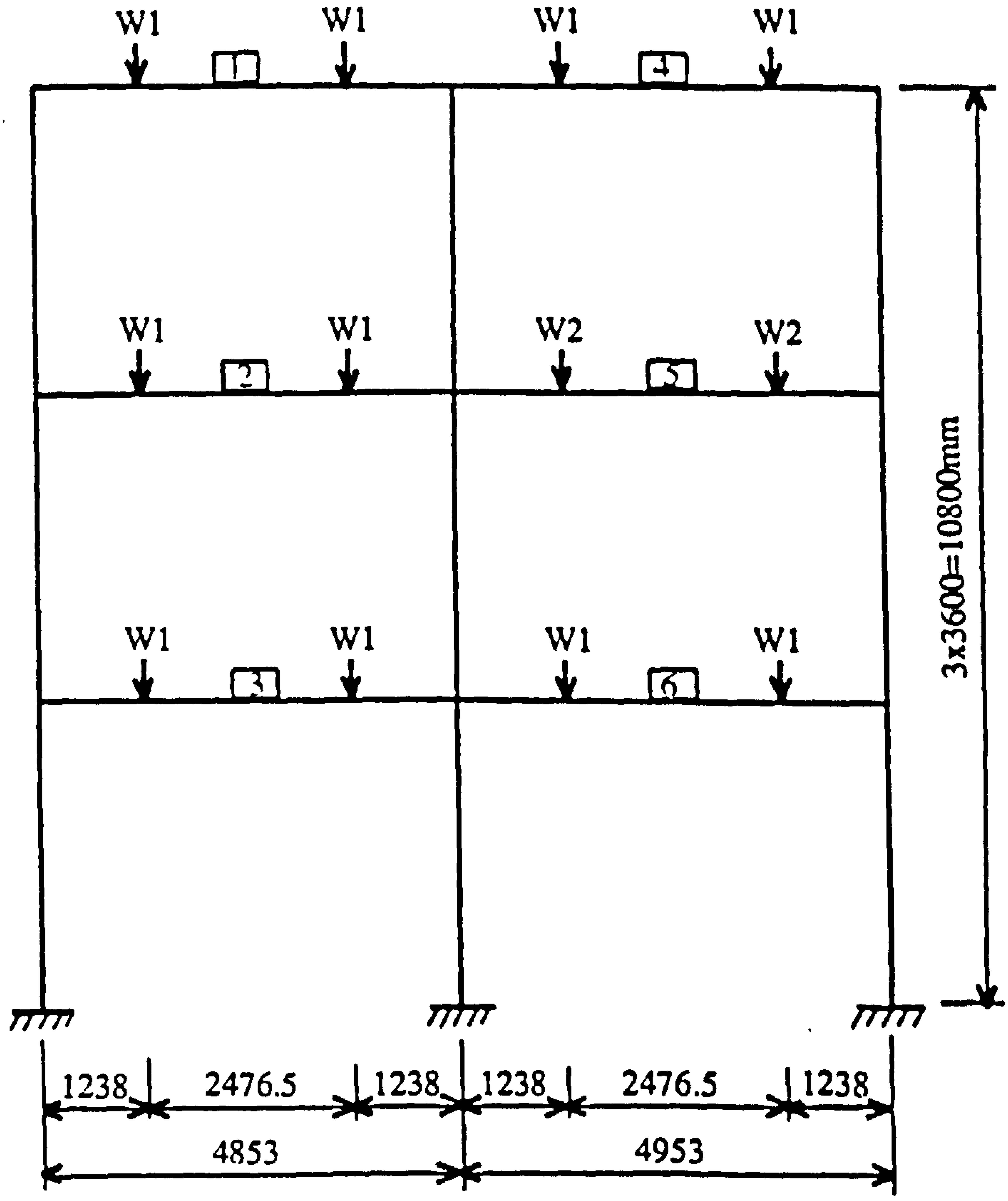
Table 7.8 Analysis results of two storey-two bay frames with 5.00 m span

Type of connection	Frame identi-fication number	Beam section "UB"	Column section "UC"		M-Φ data			Beam end moments x($wL^2/8$)			
			Outer column	Inner column	T_p (mm)	T_{fc} (mm)	D_b (mm)	Bay 1		Bay 2	
								1st end	2nd end	1st end	2nd end
Flush	FF21A	305x102x25**	152x152x30	152x152x30	8.0	9.4	304.8	16.7%	17.0%	23.1%	22.5%
end	10% rule	356x171x51**	152x152x37	203x203x46	12.5	11.5	355.6	12.9%	12.9%	17.3%	17.2%
plate	FF21B	254x102x25*	203x203x52	203x203x52	12.5	12.5	257.0	26.3%	26.9%	32.9%	32.3%
connection	20% rule	305x165x54**	203x203x71	203x203x71	20.0	17.3	310.9	28.2%	29.7%	36.5%	33.3%
Extended	FE21A	305x102x25	152x152x23	152x152x23	12.5	6.8	304.8	21.8%	30.7%	37.4%	24.7%
end	10% rule	356x171x51	152x152x30	203x203x46	12.5	9.4 11.0	355.6	15.6%	30.8%	36.8%	16.3%
plate	FE21B	254x102x25	152x152x23	152x152x23	12.5	6.8	257.0	21.0%	24.0%	30.0%	24.9%
connection	20% rule	305x165x54	203x203x46	203x203x46	15.0	11.0	310.9	22.7%	27.0%	32.3%	25.4%
Flush	FF22A	305x102x25**	152x152x30	152x152x30	8.0	9.4	304.8	16.7%	17.0%	23.7%	22.3%
end	10% rule	356x171x51**	152x152x37	203x203x46	12.5	11.5 11.0	355.6	17.1%	17.3%	12.9%	12.9%
plate	FF22B	254x102x25*	203x203x52	203x203x52	12.5	12.5	257.0	26.4%	26.5%	33.6%	32.1%
connection	20% rule	305x165x54**	203x203x71	203x203x71	20.0	17.3	310.9	33.3%	36.6%	29.6%	28.2%
Extended	FF22A	305x102x25	152x152x23	152x152x23	12.5	6.8	304.8	21.9%	30.0%	39.1%	24.4%
end	10% rule	356x171x51	152x152x30	203x203x46	12.5	9.4 11.0	355.6	16.3%	37.6%	30.5%	15.6%
plate	FE22B	254x102x25	152x152x23	152x152x23	12.5	6.8	257.0	21.2%	24.0%	30.8%	24.6%
connection	20% rule	305x165x54	203x203x46	203x203x46	15.0	11.0	310.9	25.3%	32.6%	26.8%	22.8%



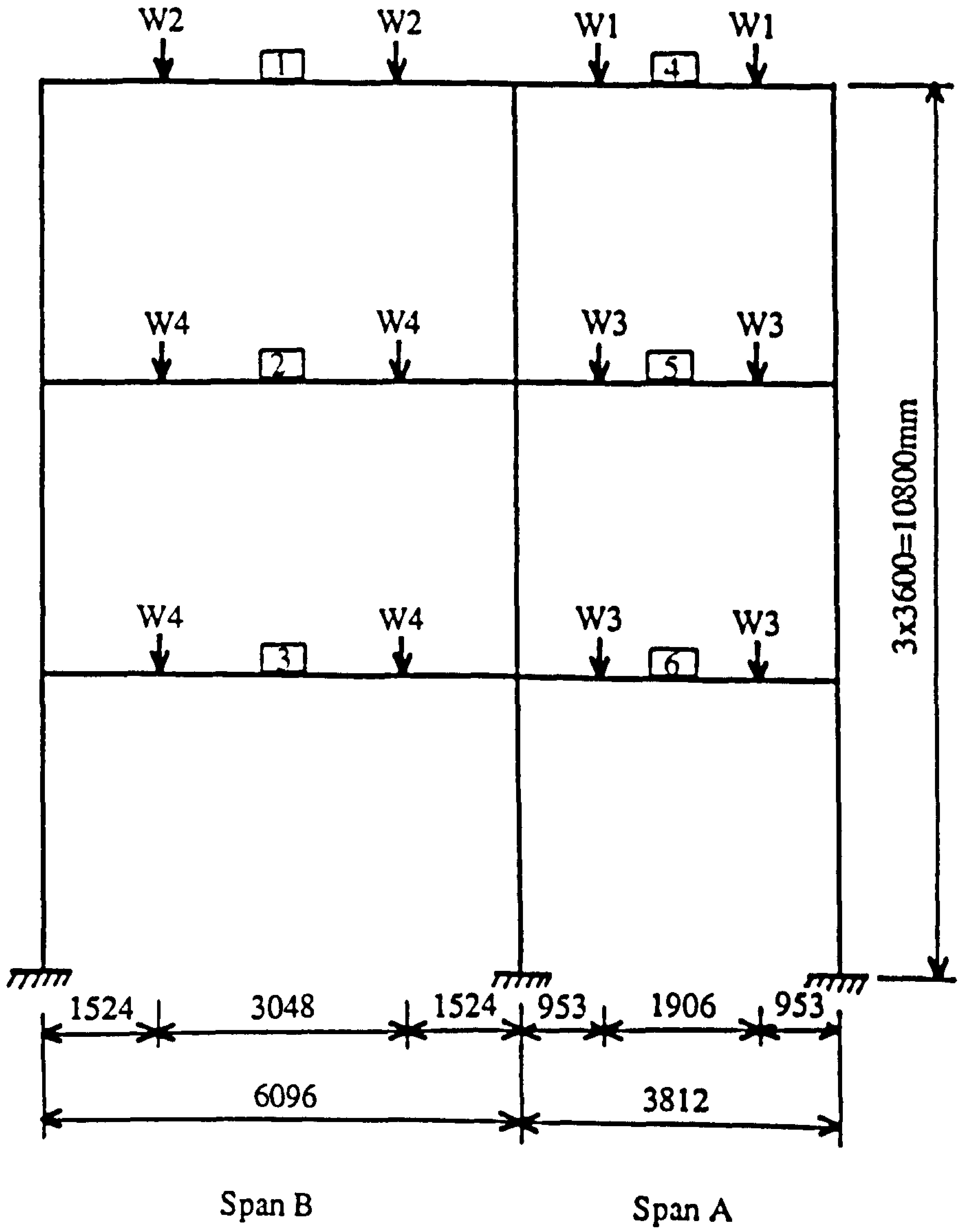
Dimensions in mm
All columns 152X152UC23
Al beams 254X102UB22
All bolts M16 grade 4.6

FIG.7.1 Geometry and loading conditions for Sheffield Frame 1



Dimensions in mm
All columns 152X152UC23
Al beams 254X102UB22
All bolts M16 grade 4.6

FIG.7.2 Geometry and loading conditions for Sheffield Frame 2



Dimensions in mm
All columns 203X203UC71
Al beams 254X146UB43
All bolts M20 grade 8.8

FIG.7.3 Geometry and loading conditions for Hatfield Frames

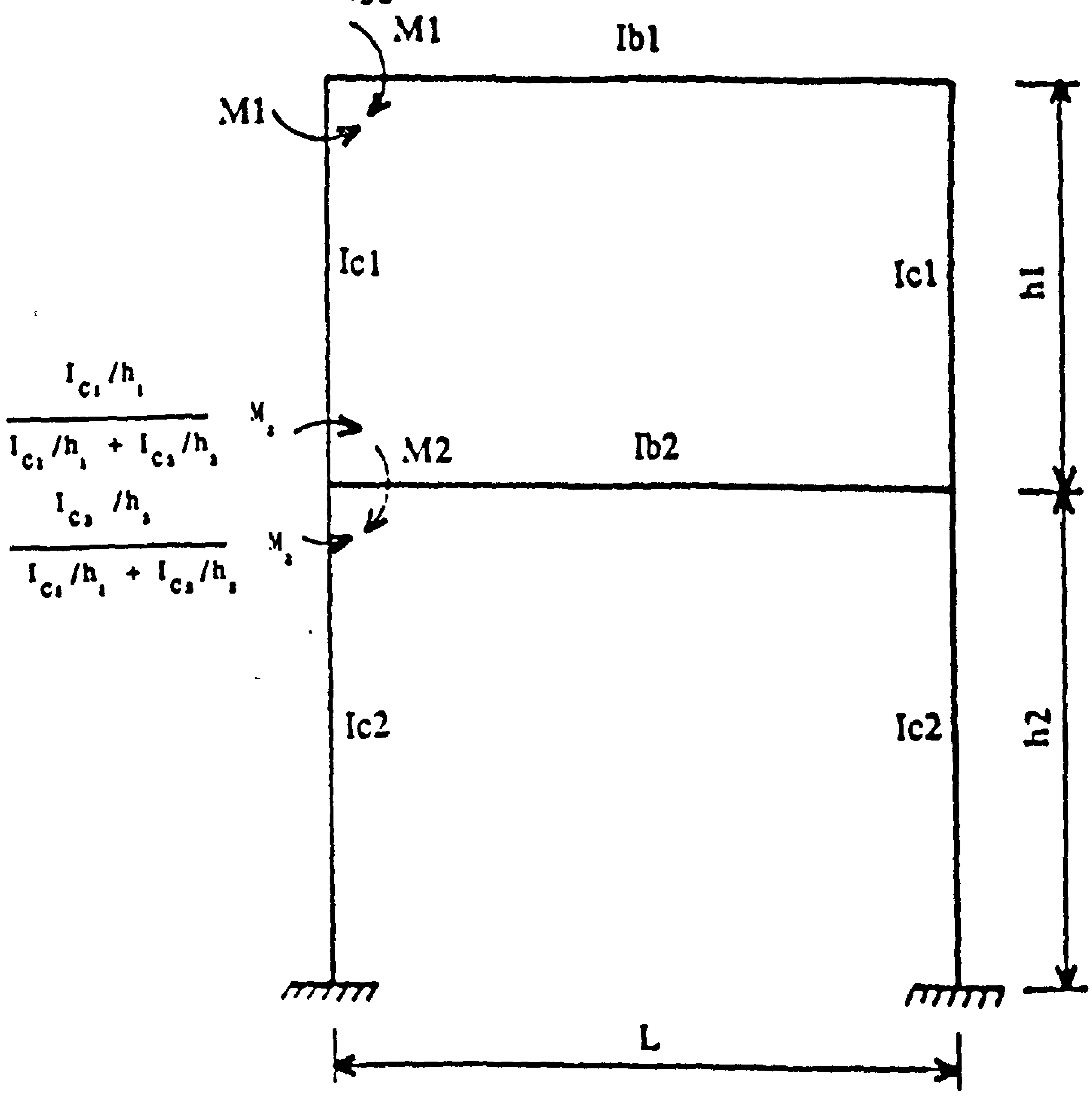
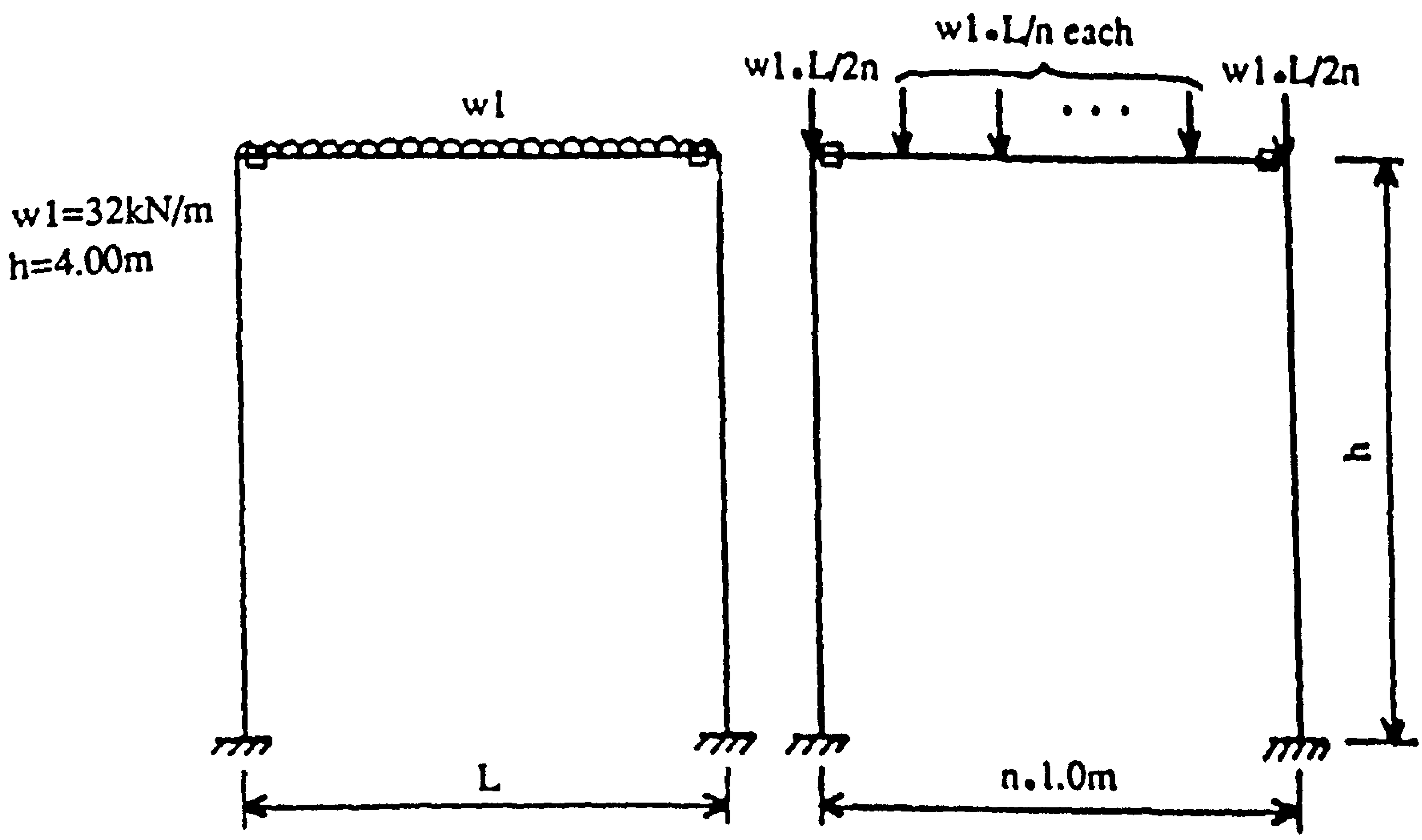


FIG.7.4 Transfer of beam end moment to the adjacent columns.



a) Frame with u.d.l.

b) Equivalent frame for analysis

FIG.7.5 Single storey-one bay frames.
(FF1A-FF6A, FF1B-FF6B, FE1A-FE6A and FE1B-FE6B).

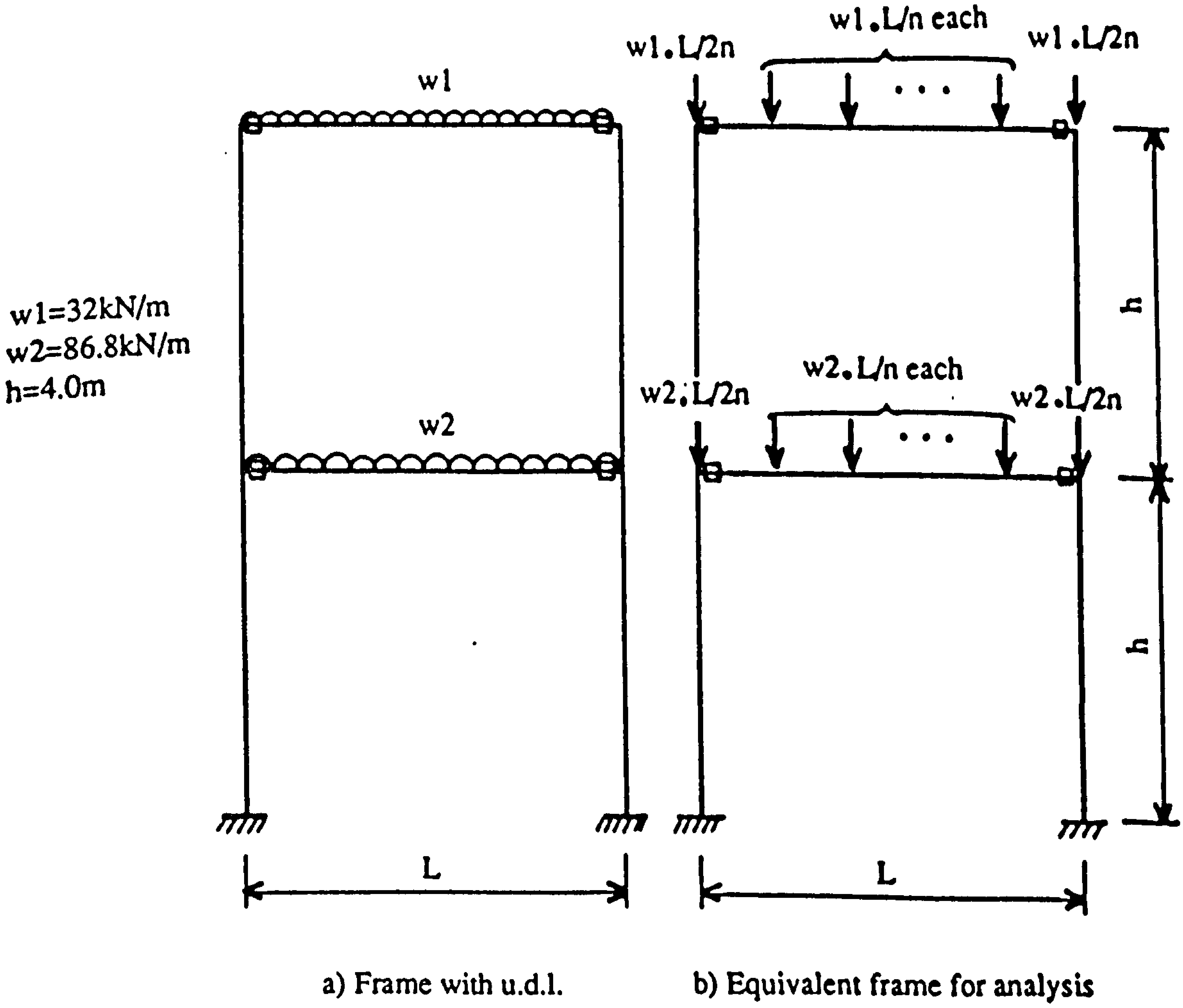
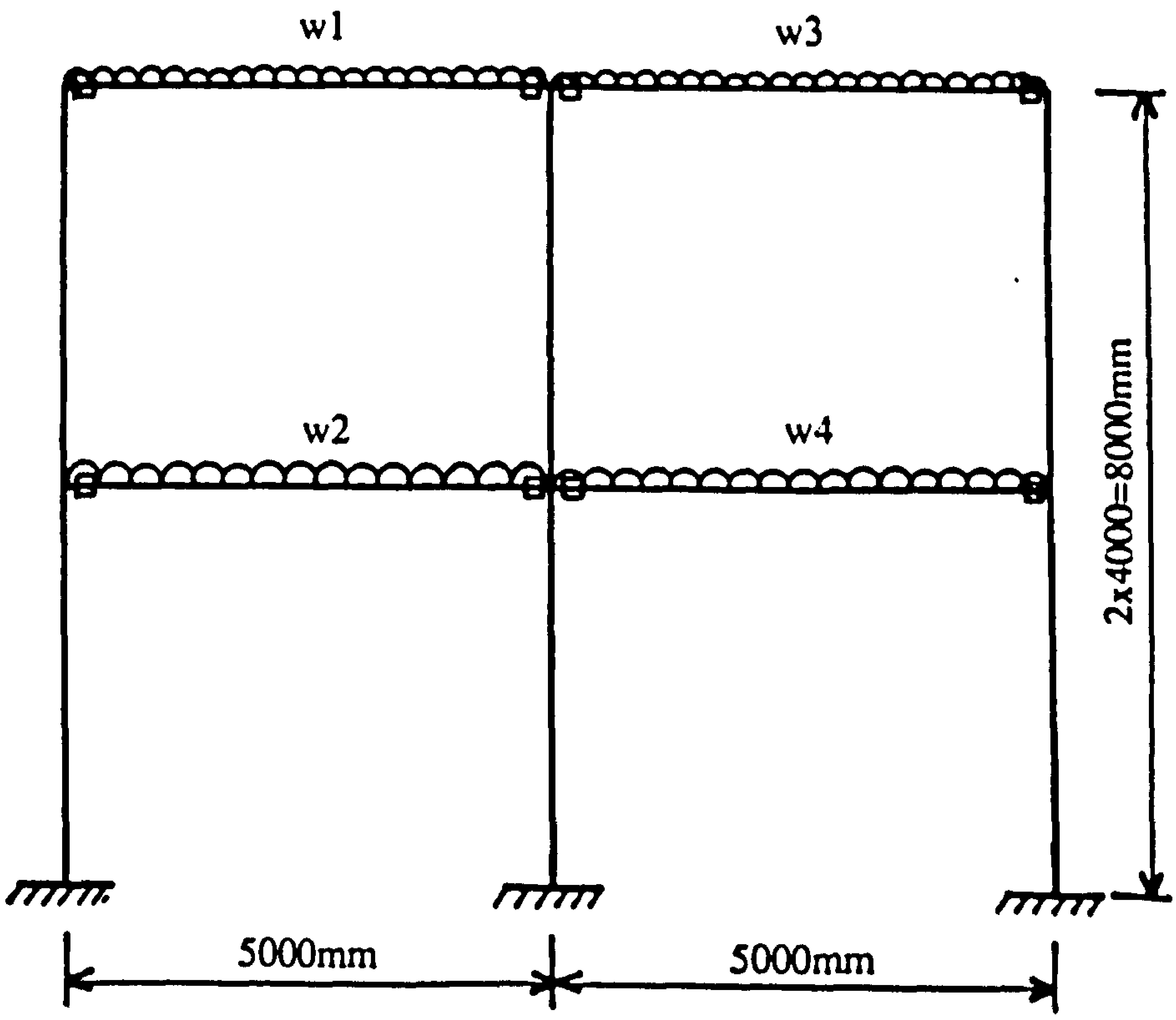
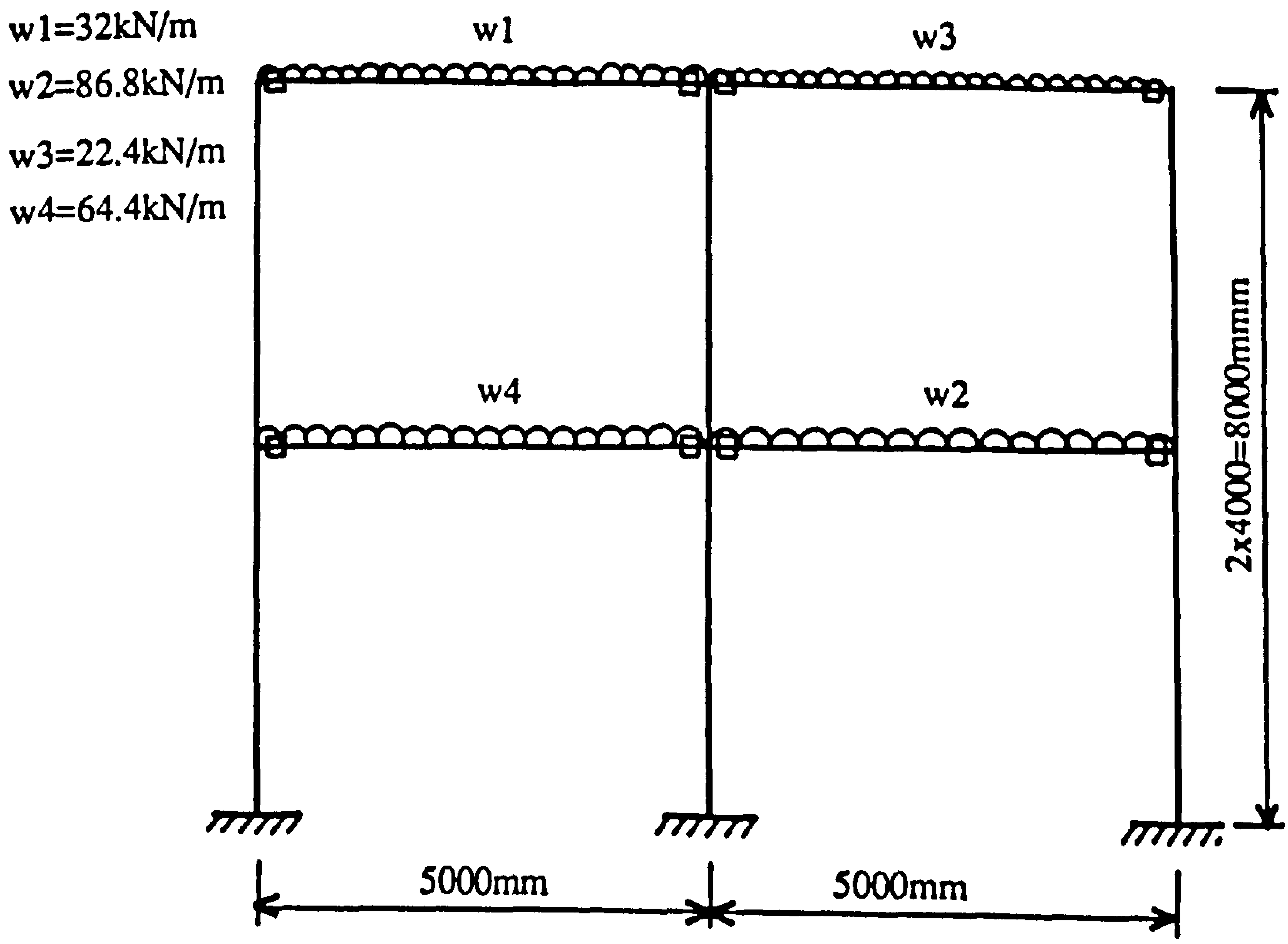


FIG.7.6 Two storey-one bay frames.
(FF7A-FF12A, FF7B-FF12B, FE7A-FE12A and FE7B-FE12B).



a) Frames FF21A, FF21B, FE21A and FE21B.



b) Frames FF22A, FF22B, FE22A and FE22B.

FIG.7.7 Geometry and loading conditions for two storey-two bay frames.

FIG.7-8 Distribution of beam end moments, flush end-plates.

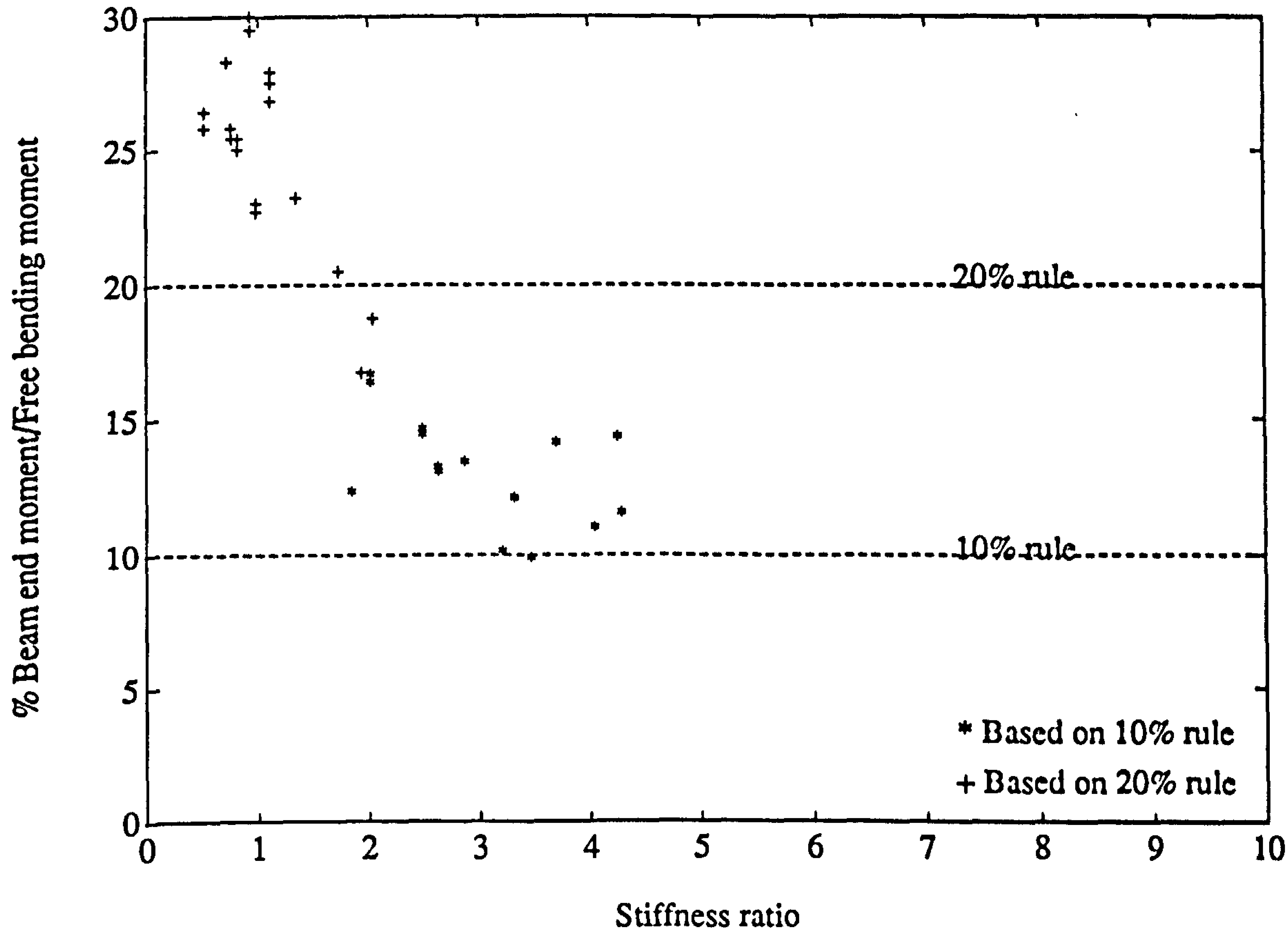
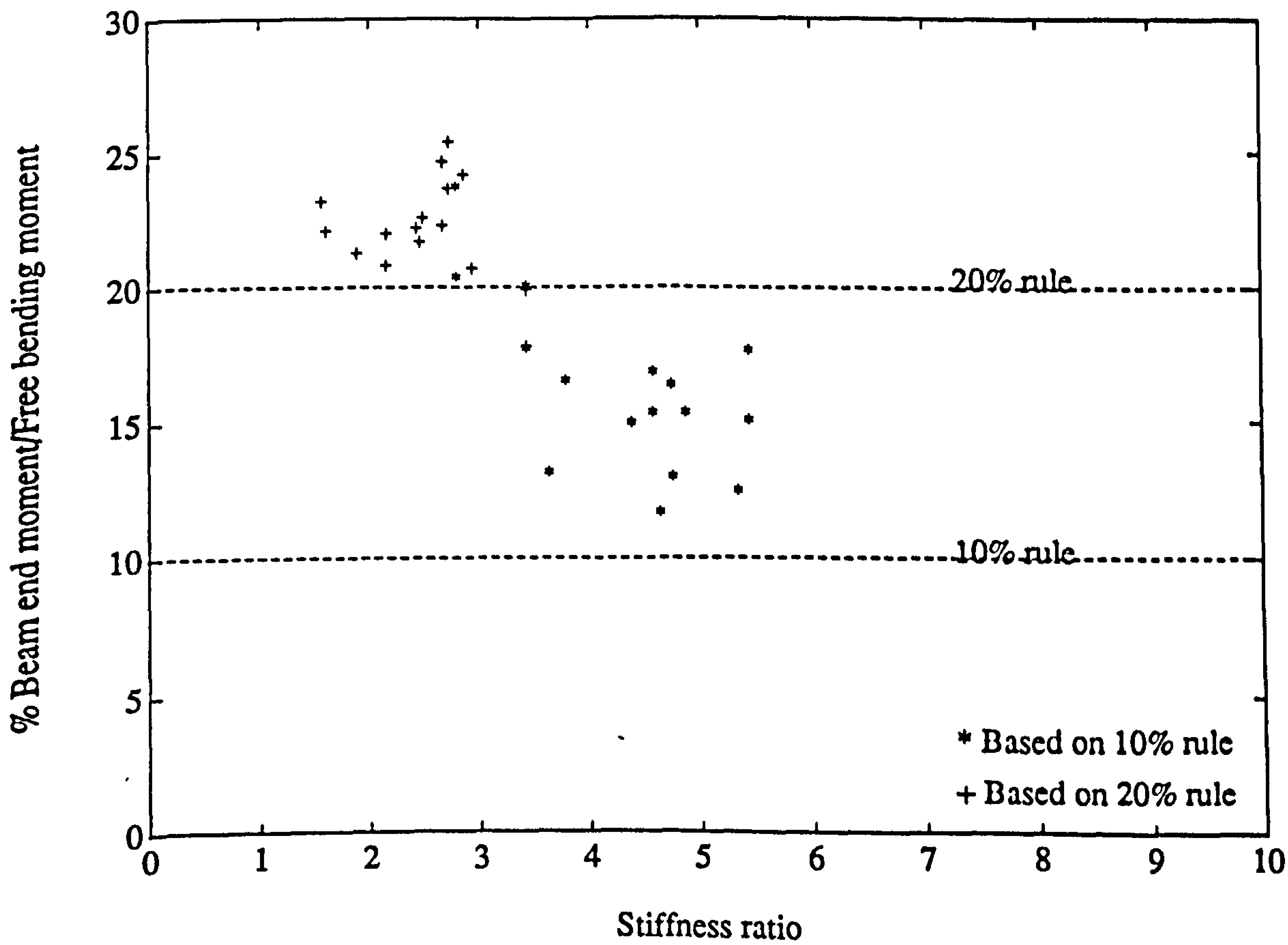
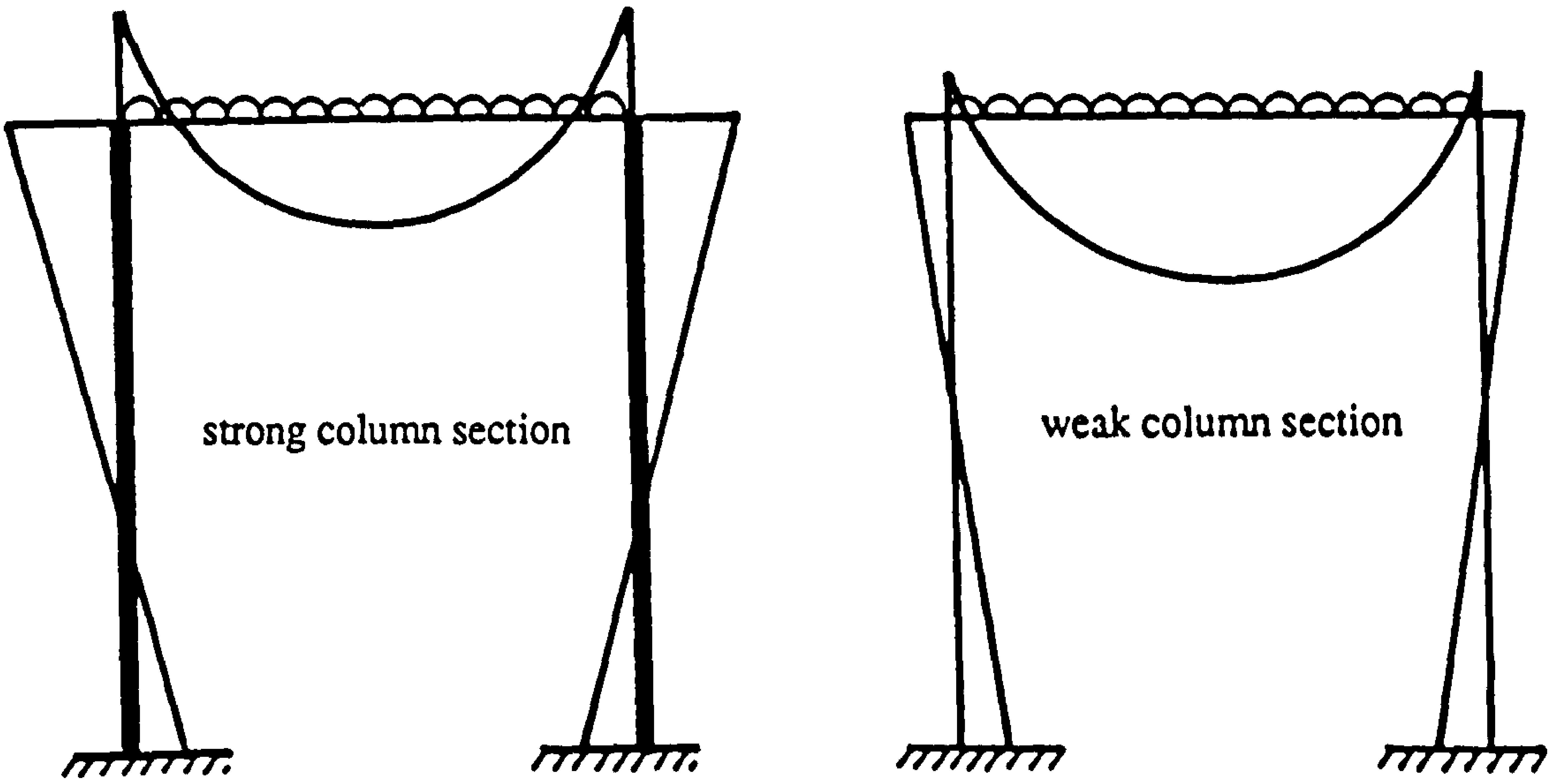


FIG.7-9 Distribution of beam end moments, extended end-plates





a) Beam ends closed to fixed ends condition for Ib.h/Σ(Ic.L) low

b) Beam ends closed to pinned ends condition for Ib.h/Σ(Ic.L) high

FIG.7.10 Scheme for the distributon of beam end moments for the two extreme cases.

CHAPTER VIII
CONCLUSIONS AND SUGGESTIONS FOR FURTHER WORK

This thesis has examined the behaviour of end-plate connections and analysis of unbraced semi-rigid steel plane frames with criteria for their design.

An extensive study has been made of available experimental data on header plate, flush and extended end-plate connections with their analytical representation. In-plane moment-rotation behaviour is an important characteristic for assessing the influence of the particular form of connection on the behaviour of members and complete frames, with the joint's M-Φ curve being the most appropriate measure of this. As seen from the results presented in previous chapters, the moment-rotation relationship is non-linear over almost the entire range of loading and the connection stiffness decreases as rotational deformation increases.

Available connection data on end-plate connections between I-section members where the beams frame into the flanges of the columns has been tabulated in Appendices A-C, and stored in a readily accessible form in the computer for future reference and use. From the data of experimental tests, the influence of joint parameters such as end-plate thickness, bolt details (including preload force), local stiffening arrangements etc., on the initial stiffness, moment capacity and failure mode of the connection and on the general M-Φ behaviour, are identified by a parametric analysis. The available experimental moment-rotation curves have been compared against analytical moment-rotation curves. The validity, advantages and disadvantages of these models have been studied and recommendations made regarding their application and use for analysis and design of semi-rigid steel frames. The equations recommended for use are identified again in the following paragraphs.

The most popular approach with researchers is a polynomial model first introduced by Sommer and then by Frye and Morris. Both models, due to Ang and Morris, and Sommer, have been presented in Chapter 3 and take into account the same connection parameters in predicting the moment-rotation curve for header plate connections. The equations are empirical and based on the data obtained by Sommer by testing a wide range of connection parameters. Both models are checked against several independent test programmes and have provided a realistic representation of connection behaviour up to rotation values of about 0.025 radians, for end-plate thickness less than 12 mm. There is no satisfactory equation available to predict the moment capacity of the connection. To achieve a close approximation of the ultimate moment capacity of the connection, the model should be based on the failure of some components of the connection and not on geometrical considerations. However, the ultimate moment capacity of a shear end-plate connection appears to be of little relevance in practice as the rotation associated with that moment is well beyond the end-rotation which will be developed in beams used in typical structures.

Flush end-plate connections are very popular in the construction industry but often do not exhibit the rigid behaviour assumed in a conventional analysis. It is essential to consider the difference in behaviour of stiffened and unstiffened connections in developing the moment-rotation relationships. The author has developed an empirical prediction equation for flush end-plate connections to unstiffened columns. The proposed prediction equation is reasonably accurate for connections with end-plate thickness greater than 6mm and for beam depths ranging from 250mm to 400mm. The experimental curves show a good agreement with the analytical curves up to a minimum connection rotation of 0.02 radians. The initial connection stiffness is reasonably well estimated. The equation predicts a higher moment capacity than that provided by the connection. There is no satisfactory prediction equation available for flush end-plate connections to stiffened columns. Further investigations are needed to determine a prediction equation for such cnnections using the data described in Chapter 4 and tabulated in Appendix B.

Extended end-plate connections are also popular and have received the widest study. Different degrees of moment transfer are expected from these connections and often they do not exhibit the rigid behaviour in a conventional analysis. The Frye and Morris prediction equations for extended end-plate connections to stiffened and unstiffened column are modified by the author and presented in Chapter 4. The equations are empirical and based on a statistical analysis of experimental data. For connections to stiffened columns, the proposed equation approximates reasonably well the behaviour for connections whose failure mode is other than bolt fracture and with a bolt preload force less than 100 kN. For connections to unstiffened columns, the modified Frye and Morris equation approximates reasonably well the experimental curves up to rotation of about 0.02 radians, after which deviation is substantial for connections whose ratio of column flange thickness to beam depth is greater or equal to 0.045.

In Chapter 5 an established computer program for second-order elastic frame analysis has been extended to frames with semi-rigid connections. Successive estimates are made of the secant stiffness of the connections, to represent their effect on frame behaviour. The non-linear M-Φ curve of the semi-rigid connection is idealised as a piece-wise linear curve. Any type of connection, pinned, rigid or semi-rigid, can be treated by the program.

The influence of joint characteristics on structural response of frames is explored in Chapter 6 by studying a number of different practical multi-storey, unbraced frames with various beam-to-column connections.

The modified Merchant-Rankine formula provided a close estimation of the non-linear elasto-plastic failure load for frames with $4 \leqslant \lambda_{cr} < 10$ and where the rigid plastic collapse mode is a combined mechanism.

These studies carried out on single storey-one bay frames under gravity loading have shown that wide variations in connection stiffness affected the elastic analysis results (i.e. deflections and moments) by less than 6% for an overestimation or an underestimation of connection rotation up to 30% of the true value.

A parameter entitled "degree of flexibility" has been introduced as a measure of the effect of semi-rigid joints on the stiffness of the frame. Within the limits of the study, it has been demonstrated that second-order effects will not be significant if the semi-rigid elastic critical load exceeds ten times the design load, and the degree of flexibility is less than 50%. This last requirement was satisfied by extended end-plate beam-to-column connections. It has been found that, under combined loading, the servicability limit on sway is likely to control design, rather than ultimate strength.

Existing proposals for semi-rigid design using limited redistribution of moment do not show significant economy over simple design. This is primarily a result of the low degree of redistribution that is permitted at present in design codes. Based on experimental results on full scale three storey-two bay steel frames with semi-rigid beam-to-column connections and based on the study carried out in Chapter 7 on several number of frames, it is recommended that the BS5950 simplified method should permit 20% end-restraint, which would improve significantly the attractiveness of the method.

The semi-rigid action in frame analysis and design needs further investigations. Several topics are identified below.

Successful application of more exact methods of semi-rigid design, based on M-Φ curves, depends on ease and accuracy with which these curves can be predicted by practitioners. No one method seems completely successful, due to uncertainties over imperfections, bolt-preload etc. Therefore research is needed on variations in practice and their influence on joint response and frame behaviour. The "γ" factors introduced in limit state design to cover uncertainties do not include at present any factor on connection stiffness, despite the variability of this property. It should be noted that underestimation of connection stiffness reduces the beam end restraint which leads to a safe beam design under gravity load but is unsafe for columns.

The traditional British approach to assume a moment due to the beam's end reaction acting at 100 mm eccentricity from column face is not included in Eurocode 3. This model is inconsistent as it is not applied to relieve the beam's sagging moment, but only to penalise the columns. As the various types of connections possess different degrees of stiffness, analyses should be made to determine typical degrees of redistribution for each type of connection. The connections examined should be substantially the same as those used in 'simple' design. The resulting end restraint moments should be used in both beam and column design, thus providing a consistent model.

Research is less advanced for minor axis connection behaviour as not many tests are available. Research should be undertaken into the behaviour of such connections and their influence on the interaction of beams and columns, so that semi-rigid design would be possible about both axes of a braced frame. In addition, test results when the column end moment reverses are important for the study of column stability.

Further potential for incorporating the semi-rigid action into frame design lies in the field of composite construction. Therefore, extension of the whole topic of semi-rigid action to frames with composite beams need to be investigated. There is little known about composite sway frames which is not even treated in the forthcoming Eurocode 4.

END-PLATE CONNECTIONS AND ANALYSIS OF SEMI-RIGID STEEL FRAMES

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APPENDICES

DATA-BASE OF END-PLATE CONNECTIONS

APPENDIX A

HEADER PLATE CONNECTIONS

Table A1 : Moment-Rotation Data : Sommer Test 5
Connection Type : Header plate connection

Tested by : Sommer, W.H. [2.3]
Test identification: Test 5

Beam size : W18 × 45
Column size : W14 × 38
End-plate : 15 × 6 × 1/4 in.

Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=1.375in.	Cc=1.500in.	G=4.000in.

Beam fastening : Full length fillet welds
Column fastening : 10 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	50.00	76.10
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	50.10	76.50

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 593 kips.in

Remarks: 1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A1 : Moment-Rotation Data : Sommer Test 5

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
1.550	0.0002
3.567	0.0004
5.575	0.0008
9.583	0.0011
13.583	0.0014
21.667	0.0027
27.667	0.0045
36.917	0.0128
43.000	0.0212
49.833	0.0332
57.000	0.0442
61.833	0.0543
63.917	0.0576
65.917	0.0601
67.083	0.0616
69.083	0.0633
71.083	0.0647
73.083	0.0661
77.917	0.0688
82.000	0.0710
88.333	0.0748
94.167	0.0790
101.667	0.0869
105.833	0.0930
108.333	0.0991

Table A2 : Moment-Rotation Data : Sommer Test 6
Connection Type : Header plate connection

Tested by : Sommer, W.H. [2.3]
Test identification: Test 6

Beam size : W24 × 76
Column size : W14 × 38
End-plate : 9 × 6 × 1/4 in.

Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=13.375in.	Cc=1.500in.	G=4.000in.

Beam fastening : Full length fillet welds
Column fastening : 6 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	51.50	77.90
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	49.30	74.60

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 195 kips.in

Remarks: 1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A2 : Moment-Rotation Data : Sommer Test 6

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
2.183	0.0025
4.600	0.0065
7.008	0.0102
11.833	0.0170
15.833	0.0234
17.583	0.0237
20.000	0.0242
23.583	0.0248
26.333	0.0250
30.333	0.0252
34.500	0.0254
41.583	0.0257
48.083	0.0260
56.083	0.0264
64.000	0.0274
72.167	0.0290
78.583	0.0313
86.667	0.0349
94.167	0.0387
97.500	0.0426
105.000	0.0461
112.500	0.0522
115.000	0.0561
111.667	0.0567
92.500	0.0583

Table A3 : Moment-Rotation Data : Sommer Test 7
Connection Type : Header plate connection

Tested by : Sommer, W.H. [2.3]
Test identification: Test 7

Beam size : W24 × 76
Column size : W14 × 38
End-plate : 12 × 6 × 1/4 in.

Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=10.375in.	Cc=1.500in.	G=4.000in.

Beam fastening : Full length fillet welds
Column fastening : 8 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	51.50	77.90
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	49.30	74.60

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 422 kips.in

Remarks: 1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A3 : Moment-Rotation Data : Sommer Test 7

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
2.192	0.0004
4.617	0.0016
7.042	0.0028
11.917	0.0041
17.583	0.0067
25.667	0.0155
28.083	0.0191
30.083	0.0205
32.083	0.0215
35.333	0.0231
39.417	0.0235
45.000	0.0239
50.667	0.0243
58.750	0.0248
66.833	0.0253
79.000	0.0261
86.667	0.0271
99.167	0.0308
107.500	0.0342
115.000	0.0376
123.333	0.0409
131.667	0.0455
134.167	0.0501
133.333	0.0537
96.667	0.0579

Table A4 : Moment-Rotation Data : Sommer Test 8
Connection Type : Header plate connection

Tested by :	Sommer, W.H. [2.3]
Test identification:	Test 8

Beam size :	W24 × 76		
Column size :	W14 × 38		
End-plate :	15 × 6 × 1/4 in.		
Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=7.375in.	Cc=1.500in.	G=4.000in.
Beam fastening :	Full length fillet welds		
Column fastening :	10 × 3/4 in. A325 bolts		
Hole type :	13/16 in. punched holes		

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	51.50	77.90
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	50.50	74.20

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 633 kips.in

Remarks: 1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A4 : Moment-Rotation Data : Sommer Test 8

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
2.200	0.0003
3.725	0.0005
6.217	0.0008
8.833	0.0011
14.333	0.0021
18.417	0.0029
22.333	0.0037
26.333	0.0050
30.333	0.0067
34.333	0.0084
40.333	0.0123
44.917	0.0165
46.417	0.0185
50.333	0.0233
52.083	0.0259
58.583	0.0279
70.667	0.0290
78.583	0.0297
84.167	0.0305
92.500	0.0311
102.500	0.0323
112.500	0.0337
116.667	0.0368
120.000	0.0415
120.000	0.0435
88.333	0.0452
85.000	0.0474
72.583	0.0496

Table A5 : Moment-Rotation Data : Sommer Test 9
Connection Type : Header plate connection

Tested by :	Sommer, W.H. [2.3]
Test identification:	Test 9

Beam size :	W24 × 76		
Column size :	W14 × 38		
End-plate :	18 × 6 × 1/4 in.		
Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=4.375in.	Cc=1.500in.	G=4.000in.
Beam fastening :	Full length fillet welds		
Column fastening :	12 × 3/4 in. A325 bolts		
Hole type :	13/16 in. punched holes		

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	51.50	77.90
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	47.40	73.10

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment:	-
Failure mode :	-
Bearing moment:	1066 kips.in

Remarks: 1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A5 : Moment-Rotation Data : Sommer Test 9

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
2.217	0.0002
4.642	0.0004
7.950	0.0006
11.167	0.0009
15.250	0.0012
20.833	0.0017
26.583	0.0024
32.917	0.0034
38.583	0.0044
44.333	0.0056
50.750	0.0072
59.000	0.0103
62.833	0.0122
70.917	0.0172
77.750	0.0232
78.167	0.0239
80.917	0.0265
85.833	0.0301
89.167	0.0333
95.000	0.0353
103.333	0.0369
111.667	0.0393
120.833	0.0420
113.333	0.0450
95.833	0.0467

Table A6 : Moment-Rotation Data : Sommer Test 10
Connection Type : Header plate connection

Tested by : Sommer, W.H. [2.3]
Test identification: Test 10

Beam size : W18 × 45
Column size : W14 × 38
End-plate : 9 × 6 × 3/8 in.
Major parameters : Lt=1.500in. Ct=1.500in. P=3.000in.
Lc=7.375in. Cc=1.500in. G=4.000in.
Beam fastening : Full length fillet welds
Column fastening : 6 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	50.00	76.10
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	57.20	75.70

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 426 kips.in

Remarks: 1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A6 : Moment-Rotation Data : Sommer Test 10

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
2.025	0.0006
5.250	0.0016
7.267	0.0022
9.250	0.0029
13.333	0.0047
15.333	0.0057
17.333	0.0070
19.333	0.0086
21.333	0.0106
23.417	0.0136
25.417	0.0182
28.667	0.0244
29.833	0.0251
31.833	0.0269
33.500	0.0278
34.250	0.0281
35.500	0.0282
36.667	0.0285
41.500	0.0289
61.667	0.0307
75.833	0.0320
84.167	0.0329
91.667	0.0349
95.833	0.0367
100.000	0.0394
104.167	0.0427
108.333	0.0464
109.167	0.0597

Table A7 : Moment-Rotation Data : Sommer Test 11
Connection Type : Header plate connection

Tested by : Sommer, W.H. [2.3]
Test identification: Test 11

Beam size : W18 × 45
Column size : W14 × 38
End-plate : 12 × 6 × 3/8 in.
Major parameters : Lt=1.500in. Ct=1.500in. P=3.000in.
Lc=4.375in. Cc=1.500in. G=4.000in.
Beam fastening : Full length fillet welds
Column fastening : 8 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	50.00	76.10
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	57.20	75.70

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 706 kips.in

Remarks: 1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A7 : Moment-Rotation Data : Sommer Test 11

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
1.575	0.0001
5.767	0.0005
6.775	0.0006
12.250	0.0012
16.167	0.0017
22.583	0.0027
26.583	0.0036
32.250	0.0052
35.500	0.0068
40.333	0.0110
42.750	0.0139
46.250	0.0198
49.167	0.0253
52.333	0.0331
56.500	0.0428
58.750	0.0469
64.500	0.0474
68.500	0.0481
73.250	0.0492
77.333	0.0499
82.167	0.0506
93.333	0.0518
106.667	0.0559
122.500	0.0609
125.000	0.0638
133.333	0.0731
127.500	0.0765

Table A8 : Moment-Rotation Data : Sommer Test 12
Connection Type : Header plate connection

Tested by : Sommer, W.H. [2.3]
Test identification: Test 12

Beam size : W18 × 45
Column size : W14 × 38
End-plate : 15 × 6 × 3/8 in.

Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=1.375in.	Cc=1.500in.	G=4.000in.

Beam fastening : Full length fillet welds
Column fastening : 10 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	50.00	76.10
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	51.60	76.00

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: -

Remarks: 1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A8 : Moment-Rotation Data : Sommer Test 12

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
1.575	0.0002
3.792	0.0003
7.817	0.0006
11.667	0.0009
15.667	0.0013
22.167	0.0021
28.167	0.0028
33.417	0.0036
41.667	0.0049
52.000	0.0071
57.000	0.0085
63.667	0.0114
72.333	0.0179
80.167	0.0279
84.167	0.0360
86.667	0.0413
84.167	0.0480
72.083	0.0538

Table A9 : Moment-Rotation Data : Sommer Test 13
Connection Type : Header plate connection

Tested by : Sommer, W.H. [2.3]
Test identification: Test 13

Beam size : W24 × 76
Column size : W14 × 28
End-plate : 9 × 6 × 3/8 in.
Major parameters : Lt=1.500in. Ct=1.500in. P=3.000in.
Lc=13.375in. Cc=1.500in. G=4.000in.
Beam fastening : Full length fillet welds
Column fastening : 6 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	51.50	77.90
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	57.20	75.70

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 425 kips.in

Remarks: 1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A9 : Moment-Rotation Data : Sommer Test 13

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
2.183	0.0009
3.783	0.0020
6.183	0.0028
10.167	0.0048
14.167	0.0073
18.167	0.0096
26.167	0.0161
35.417	0.0272
38.167	0.0273
40.167	0.0275
46.167	0.0276
54.167	0.0278
62.167	0.0282
70.167	0.0284
82.167	0.0287
90.000	0.0291
102.500	0.0294
112.500	0.0298
122.500	0.0303
132.500	0.0309
142.500	0.0318
152.500	0.0338
162.500	0.0365
172.500	0.0407
182.500	0.0462
188.333	0.0532
188.333	0.0556

Table A10 : Moment-Rotation Data : Sommer Test 14
Connection Type : Header plate connection

Tested by :	Sommer, W.H. [2.3]
Test identification:	Test 14

Beam size :	W24 × 76		
Column size :	W14 × 28		
End-plate :	12 × 6 × 3/8 in.		
Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=10.375in.	Cc=1.500in.	G=4.000in.
Beam fastening :	Full length fillet welds		
Column fastening :	8 × 3/4 in. A325 bolts		
Hole type :	13/16 in. punched holes		

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	51.50	77.90
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	57.20	75.70

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 722 kips.in

Remarks: 1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A10 : Moment-Rotation Data : Sommer Test 14

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
2.200	0.0005
4.225	0.0009
6.242	0.0013
10.250	0.0023
16.333	0.0038
22.417	0.0049
32.500	0.0070
42.583	0.0104
50.667	0.0162
58.750	0.0279
60.833	0.0312
64.417	0.0316
68.500	0.0320
76.583	0.0325
97.500	0.0335
113.333	0.0342
143.333	0.0356
154.167	0.0360
167.500	0.0368
180.000	0.0375
195.833	0.0398
212.500	0.0444
231.667	0.0553

Table A11 : Moment-Rotation Data : Sommer Test 15 Connection Type : Header plate connection

Tested by : Sommer, W.H. [2.3]
Test identification: Test 15

Beam size : $W24 \times 76$
Column size : $W14 \times 28$
End-plate : $15 \times 7 \times 3/8$ in.
Major parameters : Lt=1.500in. Ct=1.500in. P=3.000in.
Lc=7.375in. Cc=1.500in. G=5.500in.
Beam fastening : Full length fillet welds
Column fastening : $10 \times 3/4$ in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	51.50	77.90
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	58.70	75.10

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 850 kips.in

Remarks: 1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A11 : Moment-Rotation Data : Sommer Test 15

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
6.283	0.0006
10.333	0.0014
18.333	0.0027
26.417	0.0038
38.500	0.0061
50.583	0.0122
62.750	0.0283
67.833	0.0385
70.750	0.0408
74.833	0.0413
85.833	0.0427
99.167	0.0441
115.000	0.0453
139.167	0.0481
155.833	0.0515
169.167	0.0564
183.333	0.0628
193.333	0.0688
198.333	0.0752

Table A12 : Moment-Rotation Data : Sommer Test 16
Connection Type : Header plate connection

Tested by : Sommer, W.H. [2.3]
Test identification: Test 16

Beam size : W24 × 76
Column size : W14 × 28
End-plate : 18 × 7.500 × 3/8 in.
Major parameters : Lt=1.500in. Ct=1.500in. P=3.000in.
Lc=4.375in. Cc=1.500in. G=5.500in.
Beam fastening : Full length fillet welds
Column fastening : 12 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	51.50	77.90
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	61.30	75.90

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 1390 kips.in

Remarks: 1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A12 : Moment-Rotation Data : Sommer Test 16

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
2.250	0.0002
6.283	0.0004
10.333	0.0005
18.333	0.0011
26.417	0.0018
38.500	0.0029
58.667	0.0057
70.750	0.0090
86.667	0.0193
99.167	0.0325
106.667	0.0433
113.333	0.0515
117.500	0.0533
123.333	0.0547
130.833	0.0562
143.333	0.0582
155.833	0.0601
167.500	0.0623
180.000	0.0646
195.833	0.0684
211.667	0.0739
220.833	0.0816

Table A13 : Moment-Rotation Data : Sommer Test 17
Connection Type : Header plate connection

Tested by : Sommer, W.H. [2.3]
Test identification: Test 17

Beam size : W24 × 76
Column size : W14 × 28
End-plate : 12 × 7.500 × 1/4 in.
Major parameters : Lt=1.500in. Ct=1.500in. P=3.000in.
Lc=10.375in. Cc=1.500in. G=5.500in.
Beam fastening : Full length fillet welds
Column fastening : 8 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	51.50	77.90
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	49.30	74.60

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 255 kips.in

Remarks: 1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A13 : Moment-Rotation Data : Sommer Test 17

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
2.175	0.0005
6.200	0.0014
10.250	0.0062
14.250	0.0120
17.417	0.0189
19.833	0.0242
21.500	0.0256
24.667	0.0271
28.750	0.0282
32.750	0.0290
38.333	0.0297
46.417	0.0307
54.417	0.0318
66.500	0.0373
74.583	0.0428
82.583	0.0485
85.833	0.0524
88.333	0.0552
90.833	0.0583
71.333	0.0605

Table A14 : Moment-Rotation Data : Sommer Test 18
Connection Type : Header plate connection

Tested by : Sommer, W.H. [2.3]
Test identification: Test 18

Beam size : W24 × 76
Column size : W14 × 28
End-plate : 15 × 7.500 × 1/4 in.
Major parameters : Lt=1.500in. Ct=1.500in. P=3.000in.
Lc=7.375in. Cc=1.500in. G=5.500in.
Beam fastening : Full length fillet welds
Column fastening : 10 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	51.50	77.90
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	49.30	74.60

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 385 kips.in

Remarks: 1) Rotation relative to column measured
[B 2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A14 : Moment-Rotation Data : Sommer Test 18

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
2.175	0.0009
6.200	0.0028
10.250	0.0046
14.250	0.0067
19.833	0.0115
26.333	0.0203
30.333	0.0271
32.750	0.0296
35.917	0.0305
38.333	0.0311
46.417	0.0326
54.417	0.0341
62.500	0.0355
74.583	0.0407
86.667	0.0466
92.500	0.0504
102.500	0.0543
112.500	0.0583
122.500	0.0632
132.500	0.0674
142.500	0.0738
147.500	0.0893
144.167	0.0940
136.667	0.1017

Table A15 : Moment-Rotation Data : Sommer Test 19
Connection Type : Header plate connection

Tested by : Sommer, W.H. [2.3]
Test identification: Test 19

Beam size : W24 × 76
Column size : W14 × 28
End-plate : 12 × 7.500 × 1/2 in.
Major parameters : Lt=1.500in. Ct=1.500in. P=3.000in.
Lc=10.375in. Cc=5.500in. G=5.500in.
Beam fastening : Full length fillet welds
Column fastening : 8 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	51.50	77.90
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	51.10	76.10

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 702 kips.in

Remarks: 1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A15 : Moment-Rotation Data : Sommer Test 19

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
2.225	0.0004
6.267	0.0013
10.333	0.0022
22.417	0.0055
30.500	0.0093
42.667	0.0147
50.750	0.0287
54.750	0.0351
56.750	0.0401
58.833	0.0457
62.667	0.0469
64.833	0.0474
70.917	0.0479
83.083	0.0487
103.333	0.0499
119.167	0.0507
133.333	0.0515
147.500	0.0522
164.167	0.0534
174.167	0.0550
188.333	0.0584
204.167	0.0674
207.500	0.0752
209.167	0.0814

Table A16 : Moment-Rotation Data : Sommer Test 20
Connection Type : Header plate connection

Tested by :	Sommer, W.H. [2.3]
Test identification:	Test 20

Beam size :	W24 × 76		
Column size :	W14 × 28		
End-plate :	15 × 7.500 × 1/2 in.		
Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=7.375in.	Cc=1.500in.	G=5.500in.
Beam fastening :	Full length fillet welds		
Column fastening :	10 × 3/4 in. A325 bolts		
Hole type :	13/16 in. punched holes		

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	51.50	77.90
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	54.10	77.50

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 1100 kips.in

Remarks: 1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A16 : Moment-Rotation Data : Sommer Test 20

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
2.208	0.0002
6.233	0.0006
10.250	0.0009
18.333	0.0015
26.333	0.0024
38.417	0.0038
50.500	0.0061
62.500	0.0101
74.583	0.0192
82.667	0.0304
86.667	0.0372
90.000	0.0447
91.667	0.0475
94.167	0.0484
98.333	0.0489
115.000	0.0503
130.833	0.0514
146.667	0.0528
170.833	0.0552
181.667	0.0574
189.167	0.0682

Table A17: Moment-Rotation Data : Sommer Test 25
Connection Type : Header plate connection

Tested by : Sommer, W.H. [2.3]
Test identification: Test 25

Beam size : W18 × 45
Column size : W14 × 28
End-plate : 12 × 6 × 1/4 in.

Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=4.375in.	Cc=1.500in.	G=4.000in.

Beam fastening : Full length fillet welds
Column fastening : 8 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	50.00	76.1
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	49.30	74.60

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 445 kips.in

Remarks:
1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A17: Moment-Rotation Data : Sommer Test 25

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
1.550	0.0007
4.767	0.0017
8.150	0.0032
13.667	0.0060
17.667	0.0089
25.667	0.0222
29.667	0.0331
32.917	0.0419
36.917	0.0486
41.750	0.0500
49.917	0.0530
57.833	0.0560
65.917	0.0645
68.500	0.0720
59.833	0.0739

Table A18 : Moment-Rotation Data : Sommer Test 26
Connection Type : Header plate connection

Tested by :	Sommer, W.H. [2.3]
Test identification:	Test 26

Beam size :	W18 × 45		
Column size :	W14 × 28		
End-plate :	9 × 6 × 1/4 in.		
Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=7.375in.	Cc=1.500in.	G=4.000in.
Beam fastening :	Full length fillet welds		
Column fastening :	6 × 3/4 in. A325 bolts		
Hole type :	13/16 in. punched holes		

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	50.00	76.1
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	49.30	74.60

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 165 kips.in

Remarks:
1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A18 : Moment-Rotation Data : Sommer Test 26

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
1.550	0.0008
5.650	0.0025
9.583	0.0071
13.583	0.0212
15.667	0.0239
20.083	0.0262
24.083	0.0270
29.667	0.0279
38.583	0.0296
49.833	0.0406
52.167	0.0447
55.417	0.0538
54.417	0.0615
43.500	0.0625

Table A19 : Moment-Rotation Data : Sommer Test 27
Connection Type : Header plate connection

Tested by : Sommer, W.H. [2.3]
Test identification: Test 27

Beam size : W12 × 27
Column size : W14 × 28
End-plate : 9 × 6 × 1/4 in.

Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=1.500in.	Cc=1.500in.	G=4.000in.

Beam fastening : Full length fillet welds
Column fastening : 6 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	48.40	74.00
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	49.30	74.60

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 170 kips.in

Remarks: 1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A19 : Moment-Rotation Data : Sommer Test 27

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
1.142	0.0006
5.167	0.0039
9.167	0.0087
12.417	0.0175
14.000	0.0294
17.250	0.0360
21.250	0.0442
25.250	0.0503
27.250	0.0619
28.083	0.0699
28.917	0.0794
29.667	0.0947
24.667	0.1015

Table A20 : Moment-Rotation Data : Sommer Test 28
Connection Type : Header plate connection

Tested by : Sommer, W.H. [2.3]
Test identification: Test 28

Beam size : W12 × 27
Column size : W14 × 28
End-plate : 6 × 6 × 1/4 in.

Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=4.500in.	Cc=1.500in.	G=4.000in.

Beam fastening : Full length fillet welds
Column fastening : 4 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	48.40	74.00
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	49.30	74.60

Average pretension force of bolts : -
Tightening procedure: Snug + 1/2 turn

Failure moment: -
Failure mode : -
Bearing moment: 75 kips.in

Remarks: 1) Rotation relative to column measured
2) Connection subjected to shear and moment
3) Washer used under bolt head and nut

Table A20 : Moment-Rotation Data : Sommer Test 28

Moment (kips.ft)	Rotation (radians)
0.000	0.0000
1.142	0.0025
4.758	0.0229
5.725	0.0312
6.367	0.0370
8.417	0.0402
11.583	0.0427
15.583	0.0458
19.667	0.0549
22.083	0.0626
24.500	0.0737
26.083	0.0777
27.667	0.0883
30.083	0.1018
33.333	0.1207
33.333	0.1250

Table A21 : Moment-Rotation Data : Bennetts et al. Test 1
Connection Type : Header plate connection

Tested by : Bennetts,I.D., Thomas,I.R. and Grundy,P. [2.12]
Test identification: Test 1

Beam size : 610 UB 113
Column size : 250 UC 105
End-plate : 350 × 210 × 10 mm

Major parameters :	Lt=55mm	Ct=35mm	P=140mm
	Lc=205mm	Cc=35mm	G=140mm

Beam fastening : 6mm double fillet welds
Column fastening : 6 × M20 grade 8.8 bolts
Hole type : 22mmm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	444	-
Column flange:	-	-
Column web:	-	-
End-plate:	299	-
Bolts:	-	-

Average pretension force of bolts : -
Tightening procedure: Fully tensioned but used in bearing mode

Failure moment: 16.0kN.m
Failure mode : Crack of bottom end-plate weld and all bolts sheared
Bearing moment: -

Remarks: 1) Relative beam-column rotation measured

Table A21 : Moment-Rotation Data : Bennetes et al. Test 1a

Moment (kN.m)	Rotation (radians)
0.00	0.00000
4.00	0.00008
8.00	0.00013
12.00	0.00031
16.00	0.00052
16.80	0.00057
20.00	0.00202
20.90	0.00249
21.30	0.00311
21.50	0.00363
20.00	0.00388
17.50	0.00412
16.90	0.00451
16.00	0.00461

Table A22 : Moment-Rotation Data : Bennetts et al. Test 3
Connection Type : Header plate connection

Tested by : Bennetts,I.D., Thomas,I.R. and Grundy,P. [2.12]
Test identification: Test 3

Beam size : 410 UB 54
Column size : 250 UC 105
End-plate : 210 × 210 × 10 mm
Major parameters : Lt=55mm Ct=35mm P=140mm
Lc=145mm Cc=35mm G=140mm
Beam fastening : 6mm double fillet welds
Column fastening : 4 × M20 grade 8.8 bolts
Hole type : 22mmm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	366	-
Column flange:	-	-
Column web:	-	-
End-plate:	299	-
Bolts:	-	-

Average pretension force of bolts : -
Tightening procedure: Fully tensioned but used in bearing mode

Failure moment: 7.20kN.m
Failure mode : Crack in the web of beam adjacent to bottom end of the weld
Bearing moment: -

Remarks: 1) Relative beam-column rotation measured

Table A22 : Moment-Rotation Data : Bennetts et al. Test 3a

Moment (kN.m)	Rotation (radians)
0.00	0.00000
2.80	0.00018
3.60	0.00042
6.30	0.00071
8.80	0.00166
9.20	0.00311
8.50	0.00417
9.20	0.00544
8.30	0.00596
8.90	0.00700
7.70	0.00767
7.30	0.01011

Table A22 : Moment-Rotation Data : Bennetts et al. Test 3b

Moment (kN.m)	Rotation (radians)
0.00	0.00256
0.45	0.00326
3.45	0.00539
4.00	0.00591
3.70	0.00612
5.00	0.00643
4.40	0.00685
2.70	0.00715
3.30	0.00746
1.80	0.00783
0.95	0.00798

Table A23 : Moment-Rotation Data : Pham and Mansell Test F1
Connection Type : Header plate connection

Tested by : Pham,L., and Mansell,D.S. [2.13,2.14]
Test identification: Test F1

Beam size : 250 UB 31
Column size : 310 UC 198
End-plate : 140 × 150 × 8 mm

Major parameters :	Lt=65mm	Ct=35mm	P=70mm
	Lc=45mm	Cc=35mm	G=90mm

Beam fastening : 280mm of 6mm weld
Column fastening : 4 × M20 grade 8.8 bolts
Hole type : 22mmm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	-	-

Average pretension force of bolts : -
Tightening procedure: Snug tight

Failure moment: (Max. load 185kN.m)
Failure mode : Beam lateral buckling and bearing in bolt holes
Bearing moment: -

Remarks: 1) Moment-rotation data not available
2) Connection subjected to shear and moment

Table A24 : Moment-Rotation Data : Pham and Mansell Test F2
Connection Type : Header plate connection

Tested by : Pham,L., and Mansell,D.S. [2.13,2.14]
Test identification: Test F2

Beam size : 360 UB 45
Column size : 310 UC 198
End-plate : 210 × 150 × 8 mm

Major parameters :	Lt=65mm	Ct=35mm	P=70mm
	Lc=77mm	Cc=35mm	G=90mm

Beam fastening : 420mm of 6mm weld
Column fastening : 6 × M20 grade 8.8 bolts
Hole type : 22mmm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	-	-

Average pretension force of bolts : -
Tightening procedure: Snug tight

Failure moment: (Max. load 377kN)
Failure mode : Beam lateral buckling, bearing in bolt holes and flange buckling
Bearing moment: -

Remarks: 1) Moment-rotation data not available
2) Connection subjected to shear and moment
3) For test F2b, max. load was 339kN and failure was by lateral bucling,web yiel

Table A25 : Moment-Rotation Data : Pham and Mansell Test F3
Connection Type : Header plate connection

Tested by :	Pham,L., and Mansell,D.S. [2.13,2.14]
Test identification:	Test F3

Beam size :	360 UB 45		
Column size :	310 UC 198		
End-plate :	140 × 150 × 8 mm		
Major parameters :	Lt=65mm	Ct=35mm	P=70mm
	Lc=147mm	Cc=35mm	G=90mm
Beam fastening :	280mm of 6mm weld		
Column fastening :	4 × M20 grade 8.8 bolts		
Hole type :	22mmm drilled holes		

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	-	-

Average pretension force of bolts : -
Tightening procedure: Snug tight

Failure moment:	(Max. load 264kN)
Failure mode :	Bearing in bolt holes and local flange buckling
Bearing moment:	-

Remarks:
1) Moment-rotation data not available
2) Connection subjected to shear and moment
3) For test F3b, max. load was 388kN and failure was by weld cracked ,web tear

Table A26 : Moment-Rotation Data : Pham and Mansell Test F4
Connection Type : Header plate connection

Tested by : Pham,L., and Mansell,D.S. [2.13,2.14]
Test identification: Test F4

Beam size : 460 UB 67
Column size : 310 UC 198
End-plate : 350 × 150 × 8 mm
Major parameters : Lt=65mm Ct=35mm P=70mm
Lc=39mm Cc=35mm G=90mm
Beam fastening : 704mm of 6mm weld
Column fastening : 10 × M20 grade 8.8 bolts
Hole type : 22mmm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	-	-

Average pretension force of bolts : -
Tightening procedure: Snug tight

Failure moment: (Max. load 596kN)
Failure mode : WEb yielding ,bearing in bolt holes and rig failure
Bearing moment: -

Remarks: 1) Moment-rotation data not available
2) Connection subjected to shear and moment
3) For test F4b, max. load was 619kN and failure was by combination of lateral

Table A27 : Moment-Rotation Data : Pham and Mansell Test F5
Connection Type : Header plate connection

Tested by :	Pham,L., and Mansell,D.S. [2.13,2.14]
Test identification:	Test F5

Beam size :	460 UB 67		
Column size :	310 UC 198		
End-plate :	350 × 150 × 8 mm		
Major parameters :	Lt=65mm	Ct=35mm	P=70mm
	Lc=39mm	Cc=35mm	G=90mm
Beam fastening :	704mm of 6mm weld		
Column fastening :	10 × M20 grade 8.8 bolts		
Hole type :	22mmm drilled holes		

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	-	-

Average pretension force of bolts : -
Tightening procedure: Snug tight

Failure moment:	(Max. load 683kN)
Failure mode :	Bearing failur at load point
Bearing moment:	-

Remarks: 1) Moment-rotation data not available
2) Connection subjected to shear and moment

Table A28 : Moment-Rotation Data : Pham and Mansell Test F6
Connection Type : Header plate connection

Tested by : Pham,L., and Mansell,D.S. [2.13,2.14]
Test identification: Test F6

Beam size : 460 UB 67
Column size : 310 UC 198
End-plate : 141 × 150 × 8 mm

Major parameters :	Lt=55mm	Ct=40mm	P=70mm
	Lc=258mm	Cc=40mm	G=90mm

Beam fastening : 300mm of 6mm weld
Column fastening : 4 × M20 grade 8.8 bolts
Hole type : 22mmm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	-	-

Average pretension force of bolts : -
Tightening procedure: Snug tight

Failure moment: (Max. load 282kN)
Failure mode : Web yielding ,flexural yielding,lateral buckling and bearing in holes
Bearing moment: -

Remarks:
1) Moment-rotation data not available
2) Connection subjected to shear and moment
3) For test 6b,the max. load was 382kN and failure was by web yielding, deformation of end-plate and bearing in holes

Table A29 : Moment-Rotation Data : Pham and Mansell Test F7
Connection Type : Header plate connection

Tested by : Pham,L., and Mansell,D.S. [2.13,2.14]
Test identification: Test F7

Beam size : 460 UB 67
Column size : 310 UC 198
End-plate : 141 × 150 × 8 mm

Major parameters :	Lt=55mm	Ct=40mm	P=70mm
	Lc=258mm	Cc=40mm	G=90mm

Beam fastening : 300mm of 6mm weld
Column fastening : 4 × M20 grade 8.8 bolts
Hole type : 22mmm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-
Bolts:	-	-

Average pretension force of bolts : -
Tightening procedure: Snug tight

Failure moment: (Max. load 443kN)
Failure mode : Tearing of web at underside of plate and bearing in bolt holes
Bearing moment: -

Remarks: 1) Moment-rotation data not available
2) Connection subjected to shear and moment

Table A30 : Moment-Rotation Data : Hafez Test 1
Connection Type : Header plate connection

Tested by : Hafez, D.J.L. [2.4]
Test identification: Test 1

Beam size : WWF27 × 106
Column size : Heavy
End-plate : 18 × 9 × 3/8 in.
Major parameters : Lt=1.500in. Ct=1.500in. P=3.000in.
Lc=7.500in. Cc=1.500in. G=4.000in.
Beam fastening : 1/4in. double fillet weld
Column fastening : 12 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	47.40	76.50
Column flange:	-	-
Column web:	-	-
End-plate:	42.20	65.50
Bolts:	-	-

Average pretension force of bolts : Torque to 370 ft.lbs
Tightening procedure: -

Failure moment: -
Failure mode : -
Bearing moment: 1260 kips.in

Remarks: 1) Rotation relative to column measured

Table A30 : Moment-Rotation : Hafez Test 1

Moment (kips.ft)	Rotation (radians)
0.000	0.00000
19.167	0.00085
31.667	0.00140
42.500	0.00240
58.333	0.00400
74.167	0.00640
88.333	0.01050
98.333	0.01490
103.333	0.01960
105.000	0.02320
106.667	0.02728
123.333	0.02898
140.000	0.03050
157.083	0.03220
166.667	0.03330

Table A31 : Moment-Rotation Data : Hafez Test 2
Connection Type : Header plate connection

Tested by :	Hafez, D.J.L. [2.4]
Test identification:	Test 2

Beam size :	WWF27 × 106		
Column size :	Heavy		
End-plate :	18 × 9 × 1/4 in.		
Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=7.500in.	Cc=1.500in.	G=4.000in.
Beam fastening :	1/4in. double fillet weld		
Column fastening :	12 × 3/4 in. A325 bolts		
Hole type :	13/16 in. punched holes		

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	47.40	76.50
Column flange:	-	-
Column web:	-	-
End-plate:	52.50	78.50
Bolts:	-	-

Average pretension force of bolts : Torque to 370 ft.lbs
Tightening procedure: -

Failure moment: -
Failure mode : -
Bearing moment: 940 kips.in

Remarks: 1) Rotation relative to column measured

Table A31 : Moment-Rotation : Hafez Test 2

Moment (kips.ft)	Rotation (radians)
0.000	0.00000
11.250	0.00080
24.167	0.00220
37.500	0.00380
51.667	0.00640
61.667	0.01000
66.667	0.01350
68.333	0.01710
70.833	0.02000
75.000	0.02500
77.542	0.02910
100.000	0.03220
112.500	0.03500
118.333	0.04000
114.167	0.04250
100.000	0.04400
90.000	0.04480

Table A32 : Moment-Rotation Data : Hafez Test 3
Connection Type : Header plate connection

Tested by : Hafez, D.J.L. [2.4]
Test identification: Test 3

Beam size : WWF27 × 106
Column size : Heavy
End-plate : 18 × 10.50 × 3/8 in.

Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=7.500in.	Cc=1.500in.	G=5.500in.

Beam fastening : 1/4in. double fillet weld
Column fastening : 12 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	47.40	76.50
Column flange:	-	-
Column web:	-	-
End-plate:	42.20	65.50
Bolts:	-	-

Average pretension force of bolts : Torque to 370 ft.lbs
Tightening procedure: -

Failure moment: -
Failure mode : -
Bearing moment: 900 kips.in

Remarks: 1) Rotation relative to column measured

Table A32 : Moment-Rotation : Hafez Test 3

Moment (kips.ft)	Rotation (radians)
0.000	0.00000
20.833	0.00100
33.333	0.00250
46.667	0.00500
56.667	0.01000
62.500	0.01500
66.667	0.02000
68.750	0.02500
70.000	0.03000
74.583	0.03500
77.500	0.04000
100.000	0.04200
115.000	0.04300
133.333	0.04400
142.917	0.04500
166.667	0.04800

Table A33 : Moment-Rotation Data : Hafez Test 4
Connection Type : Header plate connection

Tested by :	Hafez, D.J.L. [2.4]
Test identification:	Test 4

Beam size :	WWF27 × 106		
Column size :	Heavy		
End-plate :	12 × 10.50 × 1/4 in.		
Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=13.500in.	Cc=1.500in.	G=5.500in.
Beam fastening :	1/4in. double fillet weld		
Column fastening :	8 × 3/4 in. A325 bolts		
Hole type :	13/16 in. punched holes		

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	47.40	76.50
Column flange:	-	-
Column web:	-	-
End-plate:	52.50	78.50
Bolts:	-	-

Average pretension force of bolts : Torque to 370 ft.lbs
Tightening procedure: -

Failure moment: -
Failure mode : -
Bearing moment: 225 kips.in

Remarks: 1) Rotation relative to column measured

Table A33 : Moment-Rotation : Hafez Test 4

Moment (kips.ft)	Rotation (radians)
0.000	0.00000
5.000	0.00160
9.583	0.00450
12.500	0.00810
14.167	0.01000
15.000	0.01300
15.833	0.01600
17.500	0.01900
18.333	0.02300
19.167	0.02700
20.000	0.03050
20.417	0.03200
27.917	0.03300
46.667	0.03400
55.833	0.03460
64.583	0.03550
72.500	0.03800
77.500	0.03900
80.000	0.04150

Table A34 : Moment-Rotation Data : Hafez Test 5
Connection Type : Header plate connection

Tested by : Hafez, D.J.L. [2.4]
Test identification: Test 5

Beam size :	WWF27 × 106		
Column size :	Heavy		
End-plate :	12 × 10.50 × 3/8 in.		
Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=13.500in.	Cc=1.500in.	G=5.500in.
Beam fastening :	1/4in. double fillet weld		
Column fastening :	8 × 3/4 in. A325 bolts		
Hole type :	13/16 in. punched holes		

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	47.40	76.50
Column flange:	-	-
Column web:	-	-
End-plate:	42.20	65.50
Bolts:	-	-

Average pretension force of bolts : Torque to 370 ft.lbs
Tightening procedure: -

Failure moment: -
Failure mode : -
Bearing moment: 365 kips.in

Remarks: 1) Rotation relative to column measured

Table A34 : Moment-Rotation : Hafez Test 5

Moment (kips.ft)	Rotation (radians)
0.000	0.00000
10.833	0.00150
17.500	0.00360
21.667	0.00660
23.333	0.00960
25.000	0.01200
25.833	0.01550
26.667	0.01850
27.500	0.02130
28.333	0.02450
30.000	0.02700
30.833	0.03000
41.667	0.03150
53.333	0.03200
63.333	0.03300
75.000	0.03350
86.667	0.03420
100.000	0.03500
112.500	0.03680
119.167	0.03930
126.667	0.04200
133.333	0.04500
139.167	0.04780
141.667	0.04900

Table A35 : Moment-Rotation Data : Hafez Test 6
Connection Type : Header plate connection

Tested by :	Hafez, D.J.L. [2.4]
Test identification:	Test 6

Beam size :	WWF27 × 106		
Column size :	Heavy		
End-plate :	12 × 10.50 × 1/2 in.		
Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=13.500in.	Cc=1.500in.	G=5.500in.
Beam fastening :	1/4in. double fillet weld		
Column fastening :	8 × 3/4 in. A325 bolts		
Hole type :	13/16 in. punched holes		

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	47.40	76.50
Column flange:	-	-
Column web:	-	-
End-plate:	42.30	64.10
Bolts:	-	-

Average pretension force of bolts : Torque to 370 ft.lbs
Tightening procedure: -

Failure moment: -
Failure mode : -
Bearing moment: 490 kips.in

Remarks: 1) Rotation relative to column measured

Table A35 : Moment-Rotation : Hafez Test 6

Moment (kips.ft)	Rotation (radians)
0.000	0.00000
12.900	0.00125
22.080	0.00480
28.750	0.00900
31.667	0.01260
32.017	0.01600
34.167	0.02000
35.833	0.02300
37.500	0.02700
38.750	0.03100
39.167	0.03300
57.500	0.03550
70.000	0.03650
80.833	0.03750

Table A36 : Moment-Rotation Data : Hafez Test 7
Connection Type : Header plate connection

Tested by : Hafez, D.J.L. [2.4]
Test identification: Test 7

Beam size : WWF27 × 106
Column size : Heavy
End-plate : 24 × 10.50 × 3/8 in.
Major parameters : Lt=1.500in. Ct=1.500in. P=3.000in.
Lc=1.500in. Cc=1.500in. G=5.500in.
Beam fastening : 1/4in. double fillet weld
Column fastening : 16 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	47.40	76.50
Column flange:	-	-
Column web:	-	-
End-plate:	42.20	65.50
Bolts:	-	-

Average pretension force of bolts : Torque to 370 ft.lbs
Tightening procedure: -

Failure moment: -
Failure mode : -
Bearing moment: 1780 kips.in

Remarks: 1) Rotation relative to column measured

Table A36 : Moment-Rotation : Hafez Test 7

Moment (kips.ft)	Rotation (radians)
0.000	0.00000
33.333	0.00070
50.000	0.00150
66.667	0.00270
83.333	0.00500
100.000	0.00850
105.833	0.01000
112.500	0.01500
120.833	0.02000
129.167	0.02500
134.167	0.03000
141.667	0.03500
148.333	0.04000
166.667	0.04300
175.000	0.04500
200.000	0.04920

Table A37 : Moment-Rotation Data : Hafez Test 8
Connection Type : Header plate connection

Tested by : Hafez, D.J.L. [2.4]
Test identification: Test 8

Beam size : WWF27 × 106
Column size : Heavy
End-plate : 24 × 10.50 × 1/4 in.

Major parameters :	Lt=1.500in.	Ct=1.500in.	P=3.000in.
	Lc=1.500in.	Cc=1.500in.	G=5.500in.

Beam fastening : 1/4in. double fillet weld
Column fastening : 16 × 3/4 in. A325 bolts
Hole type : 13/16 in. punched holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	47.40	76.50
Column flange:	-	-
Column web:	-	-
End-plate:	52.20	65.50
Bolts:	-	-

Average pretension force of bolts : Torque to 370 ft.lbs
Tightening procedure: -

Failure moment: -
Failure mode : -
Bearing moment: Failure before bottom beam flange bears againt column face

Remarks: 1) Rotation relative to column measured

Table A37 : Moment-Rotation : Hafez Test 8

Moment (kips.ft)	Rotation (radians)
0.000	0.00000
19.167	0.00100
33.333	0.00260
50.000	0.00500
66.667	0.00860
71.667	0.01000
83.333	0.01500
91.667	0.02000
96.667	0.02500
97.500	0.03000
95.000	0.03500
88.333	0.04000

Table A38 : Moment-Rotation Data : Zandonini and Zanon Test HP1-1
Connection Type : Header plate connection

Tested by :	Zandonini, R. and Zanon, P. [2.7]
Test identification:	Test HP1-1

Beam size :	IPE 300		
Column size :	Heavy		
End-plate :	200 × 170 × 12 mm		
Major parameters :	Lt=50mm	Ct=70mm	P=60mm
	Lc=50mm	Cc=70mm	G=105mm
Beam fastening :	Weld		
Column fastening :	4 × M20 grade 4.8 bolts		
Hole type :	22mmm drilled holes		

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	288.00	425.00
Beam web:	331.00	451.00
Column flange:	-	-
Column web:	-	-
End-plate:	356.00	475.50
Bolts:	-	-

Average pretension force of bolts : Approximately 31 KN
Tightening procedure: Snug tight up to preload 40yield strength

Failure moment:	31.30kN.m
Failure mode :	Bolts in tension
Bearing moment:	29.20 kN.m

Remarks: 1) Moment-rotation data derived from M-O curve

Table A38 : Moment-Rotation Data : Zandonini and Zanon Test HP1-1

Moment (kN.m)	Rotation (radians)
0.00	0.00000
5.00	0.00135
10.00	0.00320
15.00	0.00510
20.00	0.00955
23.50	0.02000
26.50	0.04000
28.00	0.06000
29.00	0.08000
29.40	0.12000
30.00	0.12630
31.00	0.13365

Table A39 : Moment-Rotation Data : Zandonini and Zanon Test HP1-2
Connection Type : Header plate connection

Tested by : Zandonini, R. and Zanon, P. [2.7]
Test identification: Test HP1-2

Beam size : IPE 300
Column size : Heavy
End-plate : 200 × 170 × 12 mm

Major parameters :	Lt=50mm	Ct=40mm	P=60mm
	Lc=50mm	Cc=40mm	G=105mm

Beam fastening : Weld
Column fastening : 6 × M20 grade 4.8 bolts
Hole type : 22mmm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	288.00	425.00
Beam web:	331.00	451.00
Column flange:	-	-
Column web:	-	-
End-plate:	356.00	475.50
Bolts:	-	-

Average pretension force of bolts : Approximately 31 KN
Tightening procedure: Snug tight up to preload 40yield strength

Failure moment: 58.90kN.m
Failure mode : Fracture of the end-plate
Bearing moment: 39.85 kN.m

Remarks: 1) Moment-rotation data derived from M-O curve

Table A39 : Moment-Rotation Data : Zandonini and Zanon Test HP1-2

Moment (kN.m)	Rotation (radians)
0.00	0.00000
5.00	0.00085
10.00	0.00165
15.00	0.00250
20.00	0.00455
25.00	0.00910
30.00	0.02000
33.00	0.04000
35.00	0.06000
37.00	0.08000
40.00	0.10000
45.50	0.11160
50.00	0.12000
55.00	0.12850
58.50	0.13550

Table A40 : Moment-Rotation Data : Zandonini and Zanon Test HP1-3
Connection Type : Header plate connection

Tested by :	Zandonini, R. and Zanon, P. [2.7]
Test identification:	Test HP1-3

Beam size :	IPE 300		
Column size :	Heavy		
End-plate :	244 × 170 × 12 mm		
Major parameters :	Lt=28mm	Ct=32mm	P=60mm
	Lc=28mm	Cc=32mm	G=105mm
Beam fastening :	Weld		
Column fastening :	8 × M16 grade 6.8 bolts		
Hole type :	18mmm drilled holes		

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	288.00	425.00
Beam web:	331.00	451.00
Column flange:	-	-
Column web:	-	-
End-plate:	356.00	475.50
Bolts:	-	-

Average pretension force of bolts : Approximately 29.50 KN
Tightening procedure: Snug tight up to preload 40yield strength

Failure moment:	53.80kN.m
Failure mode :	Bolt in tension
Bearing moment:	Failure before beam contacts column

Remarks: 1) Moment-rotation data derived from M-O curve

Table A40 : Moment-Rotation Data : Zandonini and Zanon Test HP1-3

Moment (kN.m)	Rotation (radians)
0.00	0.00000
5.00	0.00165
10.00	0.00290
15.00	0.00415
20.00	0.00495
25.00	0.00600
30.00	0.00745
35.00	0.01030
40.00	0.01570
43.00	0.02000
48.50	0.04000
50.00	0.04620
52.50	0.06000
53.50	0.07710

Table A41 : Moment-Rotation Data : MacIntyre Test U-3-10-101-310
Connection Type : Header plate connection

Tested by : MacIntyre, J. [2.16]
Test identification: Test U-3-10-101-310

Beam size : W310 ×45 (W12 ×31)
Column size : W360 ×64
End-plate : 304 ×165×10mm (12×6.50×2/5in.)
Major parameters : Lt=38mm Ct=38mm P=76mm
Lc=44mm Cc=38mm G=101mm
Beam fastening : 5mm (5/16in.) fillet weld
Column fastening : 6 × 3/4in. (19mm) A325 bolts
Hole type : 21mm (13/16in.) drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	440.00	511.00
Column flange:	-	-
Column web:	-	-
End-plate:	283.00	434.00
Bolts:	-	-

Average pretension force of bolts : 133kN (373N.m torque)
Tightening procedure: Torque wrench

Failure moment: Tearing of the end-plateat the top of the connection
Failure mode : 69.6 kN.m
Bearing moment: 40.6 kN.m

Remarks: 1) Relative rotaiton between the test beam axis and column axis is measured
2) Connection subjected to shear and moment
3) Hardened steel washers were used

Table A41 : Moment-Rotation Data : MacIntyre Test U-3-10-101-310

Moment (kN.m)	Rotation (radians)
0.00	0.00000
7.60	0.00100
10.80	0.00220
17.40	0.00440
22.80	0.00940
26.00	0.01300
28.20	0.02000
30.00	0.03040
30.40	0.04000
32.60	0.04970
33.60	0.06000
34.70	0.06950
36.90	0.08000
38.50	0.08860
41.00	0.10000
43.90	0.11000
50.80	0.12000
57.50	0.12700
61.80	0.13400
64.50	0.14000
66.00	0.14700
67.00	0.15200
60.00	0.15700

Table A42 : Moment-Rotation Data : MacIntyre Test U-4-10-101-460
Connection Type : Header plate connection

Tested by : MacIntyre, J. [2.16]
Test identification: Test U-4-10-101-460

Beam size :	W460 ×67 (W8 ×45)		
Column size :	W360 ×64		
End-plate :	304 ×165×10mm (12×6.50×2/5in.)		
Major parameters :	Lt=38mm	Ct=38mm	P=76mm
	Lc=118mm	Cc=38mm	G=101mm
Beam fastening :	6mm (1/4in.) fillet weld		
Column fastening :	8 × 3/4 in.(19mm) A325 bolts		
Hole type :	21mm (13/16in.) drilled holes		

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	347.00	503.00
Column flange:	-	-
Column web:	-	-
End-plate:	341.00	509.00
Bolts:	-	-

Average pretension force of bolts : 133kN (373N.m torque)
Tightening procedure: Torque wrench

Failure moment: Left hand side weld sheared horizontally from the beam web at the top of
Failure mode : 160.00 kN.m
Bearing moment: 84.20kN.m

Remarks: 1) Relative rotaiton between the test beam axis and column axis is measured
2) Connection subjected to shear and moment
3) Hardened steel washers were used

Table A42 : Moment-Rotation Data : MacIntyre Test U-4-10-101-460

Moment (kN.m)	Rotation (radians)
0.00	0.00000
17.50	0.00150
30.20	0.00280
40.00	0.00450
48.80	0.00700
56.50	0.01050
61.00	0.01450
66.00	0.02000
72.00	0.03000
76.50	0.04000
80.00	0.05100
83.00	0.06000
86.00	0.06400
106.00	0.06800
120.00	0.07100
138.50	0.07350
160.00	0.08000
148.00	0.08150

Table A43 : Moment-Rotation Data : MacIntyre Test U-4-10-101-610
Connection Type : Header plate connection

Tested by : MacIntyre, J. [2.16]
Test identification: Test U-4-10-101-610

Beam size : W610 ×113 (W24 ×76)
Column size : W360 ×64
End-plate : 304 ×165×10mm (12×6.50×2/5in.)
Major parameters : Lt=38mm Ct=38mm P=76mm
Lc=268mm Cc=38mm G=101mm
Beam fastening : 8mm (5/16in.) fillet weld
Column fastening : 8 × 3/4 in.(19mm) A325 bolts
Hole type : 21mm (13/16in.) drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	325.00	469.00
Column flange:	-	-
Column web:	-	-
End-plate:	341.00	509.00
Bolts:	-	-

Average pretension force of bolts : 133kN (373N.m torque)
Tightening procedure: Torque wrench

Failure moment: Left hand side weld sheared horizontally from the beam web at the top of
Failure mode : 248.00 kN.m
Bearing moment: 76.20kN.m

Remarks: 1) Relative rotaiton between the test beam axis and column axis is measured
2) Connection subjected to shear and moment
3) Hardened steel washers were used

Table A43 : Moment-Rotation Data : MacIntyre Test U-4-10-101-610

Moment (kN.m)	Rotation (radians)
0.00	0.00000
15.70	0.00100
24.50	0.00150
34.00	0.00300
40.00	0.00450
47.50	0.00650
55.00	0.00900
63.00	0.01350
70.00	0.02000
75.00	0.02650
78.00	0.03300
100.00	0.03550
120.00	0.03650
139.50	0.03750
160.00	0.03800
175.00	0.03950
200.00	0.04000
220.00	0.04150
240.00	0.04360
246.50	0.04600
240.00	0.04900

APPENDIX B

FLUSH END-PLATE CONNECTIONS

Table B1 : Moment-Rotation Data : Ostrander - Test 1
Connection Type : Flush end-plate connection to unstiffened column

Tested by : Ostrander, J.R. [3.4]
Test identification: Test 1

Beam size : W10 × 21
Column size : W8 × 28
End-plate : 11 × 6.5 × 0.5 in.

Major parameters :	Lb=5.000in	Ct=0.500in	P=0.000in
	Li=5.000in	Cc=0.500in	G=3.500in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners : No
Beam fastening : Weld
Column fastening : 4 × 3/4 in. diameter A325 bolts
Hole type : 13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	41.3	66.9
Beam web:	43.8	66.6
Column flange:	51.1	77.3
Column web:	50.9	77.8
End-plate:	45.4	70.3

Average pretension force of bolts : 41.7 kips (0.0202 in. elongation)

Failure moment: 550 kips-in.
Failure mode : Beam compression flange buckled

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B1 : Moment-Rotation Data : Ostrander - Test 1

Moment (Kips-in.)	Rotation (radians)
0.	0.0000
24	0.0003
47	0.0006
94	0.0011
141	0.0017
188	0.0024
235	0.0035
282	0.0051
329	0.0070
376	0.0097
423	0.0136
466	0.0201
497	0.0291
519	0.0389
548	0.0550

Table B2 : Moment-Rotation Data : Ostrander - Test 2
Connection Type : Flush end-plate connection to stiffened column

Tested by : Ostrander, J.R. [3.4]
Test identification: Test 2

Beam size : W10 × 21
Column size : W8 × 28
End-plate : 11 × 6.5 × 0.5 in.

Major parameters :	Lb=5.000in	Ct=0.500in	P=0.000in
	Li=5.000in	Cc=0.500in	G=3.500in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners : Column web stiffeners opposite both beam flanges (7.25× 3 ×0.45 in.)
Beam fastening : Weld
Column fastening : 4 × 3/4 in. diameter A325 bolts
Hole type : 13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	41.3	66.9
Beam web:	43.8	66.6
Column flange:	51.1	77.3
Column web:	50.9	77.8
End-plate:	45.4	70.3

Average pretension force of bolts : 41.7 kips (0.0192 in. elongation)

Failure moment: (Max. recoreded moment 623 kips.in)
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B2 : Moment-Rotation Data : Ostrander - Test 2

Moment (Kips.in)	Rotation (radians)
0.	0.0000
24	0.0002
47	0.0005
94	0.0010
141	0.0016
188	0.0022
235	0.0028
282	0.0037
329	0.0048
376	0.0062
423	0.0083
466	0.0111
518	0.0152
562	0.0256
593	0.0367
623	0.0504

Table B3 : Moment-Rotation Data : Ostrander - Test 3 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 3

Beam size :	W10 × 21
Column size :	W8 × 28
End-plate :	11 × 6.5 × 0.375 in.

Major parameters :	Lb=5.000in	Ct=0.500in	P=0.000in
	Li=5.000in	Cc=0.500in	G=3.500in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	41.3	66.9
Beam web:	43.8	66.6
Column flange:	51.1	77.3
Column web:	50.9	77.8
End-plate:	46.5	68.5

Average pretension force of bolts : 39.7 kips (0.0108 in. elongation)

Failure moment:	(Max. recoreded moment 486 kips.in)
Failure mode :	Failure stage not reached

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B3 : Moment-Rotation Data : Ostrander - Test 3

Moment (Kips.in)	Rotation (radians)
0.	0.0000
47	0.0005
94	0.0012
141	0.0020
188	0.0030
235	0.0044
282	0.0063
329	0.0093
376	0.0142
423	0.0233
461	0.0375
486	0.0525

Table B4 : Moment-Rotation Data : Ostrander - Test 4 Connection Type : Flush end-plate connection to unstiffened column

Tested by : Ostrander, J.R. [3.4]
Test identification: Test 4

Beam size : W10 × 21
Column size : W8 × 28
End-plate : 11 × 6.5 × 0.25 in.

Major parameters :	Lb=5.000in	Ct=0.500in	P=0.000in
	Li=5.000in	Cc=0.500in	G=3.500in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners : No
Beam fastening : Weld
Column fastening : 4 × 3/4 in. diameter A325 bolts
Hole type : 13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	41.3	66.9
Beam web:	43.8	66.6
Column flange:	51.1	77.3
Column web:	50.9	77.8
End-plate:	46.5	71.2

Average pretension force of bolts : 33 kips (0.0147 in. elongation)

Failure moment: (Max. recoreded moment 264 kips.in)
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B4 : Moment-Rotation Data : Ostrander - Test 4

Moment (Kips.in)	Rotation (radians)
0.	0.0000
47	0.0010
94	0.0021
141	0.0039
188	0.0093
212	0.0152
235	0.0226
264	0.0362

Table B5 : Moment-Rotation Data : Ostrander - Test 5
Connection Type : Flush end-plate connection to stiffened column

Tested by : Ostrander, J.R. [3.4]
Test identification: Test 5

Beam size : W10 × 21
Column size : W8 × 28
End-plate : 11 × 6.5 × 0.50 in.

Major parameters :	Lb=5.000in	Ct=0.500in	P=0.000in
	Li=5.000in	Cc=0.500in	G=3.500in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners : Column web stiffeners opposite both beam flanges (7.125×3×0.25 in.)
Beam fastening : Weld
Column fastening : 4 × 3/4 in. diameter A325 bolts
Hole type : 13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	41.3	66.9
Beam web:	43.8	66.6
Column flange:	51.1	77.3
Column web:	50.9	77.8
End-plate:	45.4	70.3

Average pretension force of bolts : 47.4 kips (0.0211 in. elongation)

Failure moment: 600 kips.in
Failure mode : Beam compression buckle

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B5 : Moment-Rotation Data : Ostrander - Test 5

Moment (Kips.in)	Rotation (radians)
0.	0.0000
47	0.0005
94	0.0010
141	0.0015
188	0.0020
235	0.0027
282	0.0036
329	0.0048
376	0.0064
423	0.0090
470	0.0129
517	0.0193
561	0.0318
598	0.0510
670	0.1020

Table B6 : Moment-Rotation Data : Ostrander - Test 6
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 6

Beam size :	W10 × 21
Column size :	W8 × 28
End-plate :	11 × 6.5 × 0.375 in.

Major parameters :	Lb=5.000in	Ct=0.500in	P=0.000in
	Li=5.000in	Cc=0.500in	G=3.500in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners :	Column web stiffeners opposite both beam flanges (7.125×3×0.25 in.)
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	41.3	66.9
Beam web:	43.8	66.6
Column flange:	51.1	77.3
Column web:	50.9	77.8
End-plate:	45.5	68.5

Average pretension force of bolts : -

Failure moment: 520 kips.in
Failure mode : Beam compression buckle and end-plate weld failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B6 : Moment-Rotation Data : Ostrander - Test 6

Moment (Kips.in)	Rotation (radians)
0.	0.0000
47	0.0003
94	0.0007
141	0.0011
188	0.0020
235	0.0031
282	0.0047
329	0.0074
376	0.0119
423	0.0188
458	0.0286
492	0.0405
520	0.0566
537	0.0734

Table B7 : Moment-Rotation Data : Ostrander - Test 7
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 7

Beam size :	W10 × 21
Column size :	W8 × 28
End-plate :	11 × 6.5 × 0.250 in.

Major parameters :	Lb=5.000in	Ct=0.500in	P=0.000in
	Li=5.000in	Cc=0.500in	G=3.500in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners :	Column web stiffeners opposite both beam flanges (7.125×3×0.250 in.)
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	41.3	66.9
Beam web:	43.8	66.6
Column flange:	51.1	77.3
Column web:	50.9	77.8
End-plate:	46.5	71.2

Average pretension force of bolts : -

Failure moment: 310 kips.in
Failure mode : End-plate weld failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B7 : Moment-Rotation Data : Ostrander - Test 7

Moment (Kips.in)	Rotation (radians)
0.	0.0000
47	0.0009
94	0.0020
141	0.0036
188	0.0076
212	0.0117
235	0.0174
258	0.0258
282	0.0346
306	0.0453
312	0.0505

Table B8 : Moment-Rotation Data : Ostrander - Test 8
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 8

Beam size :	W10 × 21
Column size :	W8 × 28
End-plate :	11 × 8.5 × 0.25 in.

Major parameters :	Lb=5.000in	Ct=0.500in	P=0.000in
	Li=5.000in	Cc=0.500in	G=3.500in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners :	Column web stiffeners opposite both beam flanges (7.125×3×0.375 in.)
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	41.3	66.9
Beam web:	43.8	66.6
Column flange:	51.1	77.3
Column web:	50.9	77.8
End-plate:	46.5	71.2

Average pretension force of bolts :-

Failure moment: 301 kips.in
Failure mode : End plate weld failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B8 : Moment-Rotation Data : Ostrander - Test 8

Moment (Kips.in)	Rotation (radians)
0.	0.0000
47	0.0005
94	0.0016
141	0.0031
188	0.0071
212	0.0112
235	0.0174
258	0.0261
282	0.0349
301	0.0444

Table B9 : Moment-Rotation Data : Ostrander - Test 9
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 9

Beam size :	W10 × 21
Column size :	W8 × 28
End-plate :	11 × 6.5 × 0.750 in.

Major parameters :	Lb=5.000in	Ct=0.500in	P=0.000in
	Li=5.000in	Cc=0.500in	G=3.500in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	42.8	48.2
Beam web:	48.2	71.1
Column flange:	45.9	74.9
Column web:	51.0	76.6
End-plate:	32.00	58.5

Average pretension force of bolts : 42.1 kips (0.0242 in. elongation)

Failure moment: 600 kips.in
Failure mode : Beam compression flange buckle and column web buckle

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B9 : Moment-Rotation Data : Ostrander - Test 9

Moment (Kips.in)	Rotation (radians)
0.	0.0000
47	0.0003
94	0.0005
141	0.0009
188	0.0013
235	0.0018
282	0.0024
329	0.0035
376	0.0052
417	0.0076
466	0.0124
520	0.0211
557	0.0365
600	0.0591

Table B10 : Moment-Rotation Data : Ostrander - Test 10
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 10

Beam size :	W10 × 21
Column size :	W8 × 28
End-plate :	11 × 6.5 × 0.750 in.

Major parameters :	Lb=5.000in	Ct=0.500in	P=0.000in
	Li=5.000in	Cc=0.500in	G=3.500in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners :	Column web stiffeners opposite both beam flanges (7.125×0.375×0.375 in.)
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	42.8	48.2
Beam web:	48.2	71.1
Column flange:	45.9	74.9
Column web:	51.0	76.6
End-plate:	32.00	58.5

Average pretension force of bolts : 41.7 kips (0.0184 in. elongation)

Failure moment: 740 kips.in
Failure mode : Bolt failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table 10 : Moment-Rotation Data : Ostrander - Test 10

Moment (Kips.in)	Rotation (radians)
0.	0.0000
47	0.0001
94	0.0004
141	0.0007
188	0.0010
235	0.0013
282	0.0017
329	0.0023
376	0.0029
423	0.0038
470	0.0052
517	0.0072
561	0.0099
606	0.0146
651	0.0212
696	0.0337
743	0.0483

Table B11 : Moment-Rotation Data : Ostrander - Test 11
Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 11

Beam size :	W10 × 21
Column size :	W8 × 40
End-plate :	13 × 7.5 × 0.375 in.

Major parameters :	Lb=7.000in	Ct=0.500in	P=0.000in
	Li=7.000in	Cc=0.500in	G=4.000in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	
Stiffeners :	No		
Beam fastening :	Weld		
Column fastening :	4 × 3/4 in. diameter A325 bolts		
Hole type :	13/16 in. column holes drilled and end-plate punched		

		Yield Stress (ksi)	Ultimate Stress (ksi)
Measured Material Properties:	Beam flange:	58.0	78.8
	Beam web:	64.2	82.0
	Column flange:	42.0	71.6
	Column web:	48.6	75.0
	End-plate:	62.9	109.1

Average pretension force of bolts : 41.5 kips (0.0177 in. elongation)

Failure moment: 640 kips.in
Failure mode : End plate weld failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B11 : Moment-Rotation Data : Ostrander - Test 11

Moment (Kips.in)	Rotation (radians)
0.	0.0000
47	0.0004
94	0.0008
141	0.0013
188	0.0020
235	0.0027
282	0.0035
329	0.0045
376	0.0061
418	0.0086
468	0.0128
512	0.0204
550	0.0279
609	0.0446
639	0.0534

Table B12 : Moment-Rotation Data : Ostrander - Test 12 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 12

Beam size :	W12 × 27
Column size :	W8 × 40
End-plate :	13 × 7.5 × 0.25 in.

Major parameters :	Lb=7.000in	Ct=0.500in	P=0.000in
	Li=7.000in	Cc=0.500in	G=4.000in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	58.0	78.8
Beam web:	64.2	82.0
Column flange:	42.0	71.6
Column web:	48.6	75.0
End-plate:	42.4	71.1

Average pretension force of bolts :37.4 kips (0.0080 in. elongation)

Failure moment:	810 kips.in
Failure mode :	Bolt failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B12 : Moment-Rotation Data : Ostrander - Test 12

Moment (Kips.in)	Rotation (radians)
0.	0.0000
94	0.0005
188	0.0011
282	0.0017
376	0.0025
470	0.0040
517	0.0056
565	0.0081
611	0.0111
656	0.0157
700	0.0223
745	0.0301
780	0.0446
807	0.0522

Table B13 : Moment-Rotation Data : Ostrander - Test 13 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 13

Beam size :	W12 × 27
Column size :	W8 × 40
End-plate :	13 × 7.5 × 0.625 in.

Major parameters :	Lb=7.000in	Ct=0.500in	P=0.000in
	Li=7.000in	Cc=0.500in	G=4.000in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	58.0	78.8
Beam web:	64.2	82.0
Column flange:	42.0	71.6
Column web:	48.6	75.0
End-plate:	40.9	73.1

Average pretension force of bolts :41.7 kips (0.0216 in. elongation)

Failure moment:	900 kips.in
Failure mode :	Bolt failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B13 : Moment-Rotation Data : Ostrander - Test 13

Moment (Kips.in)	Rotation (radians)
0.	0.0000
94	0.0008
188	0.0018
282	0.0028
376	0.0041
470	0.0057
564	0.0080
611	0.0096
655	0.0114
705	0.0140
749	0.0178
790	0.0218
836	0.0290
886	0.0384

Table B14 : Moment-Rotation Data : Ostrander - Test 14
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 14

Beam size :	W12 × 27
Column size :	W8 × 40
End-plate :	13 × 7.5 × 0.375 in.

Major parameters :	Lb=7.000in	Ct=0.500in	P=0.000in
	Li=7.000in	Cc=0.500in	G=4.000in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners :	Column web stiffners opposite both beam flanges (7 × 4 × 0.250 in.)
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	58.0	78.8
Beam web:	64.2	82.0
Column flange:	42.0	71.6
Column web:	48.6	75.0
End-plate:	60.9	109.1

Average pretension force of bolts :35.4 kips (0.0069 in. elongation)

Failure moment:	715 kips.in
Failure mode :	Bolt failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B14 : Moment-Rotation Data : Ostrander - Test 14

Moment (Kips.in)	Rotation (radians)
0.	0.0000
47	0.0002
141	0.0022
235	0.0040
329	0.0060
423	0.0087
470	0.0111
517	0.0146
559	0.0191
583	0.0239
608	0.0277
653	0.0368
700	0.0467
714	0.0530

Table B15 : Moment-Rotation Data : Ostrander - Test 15
Connection Type : Flush end-plate connection to stiffened column

Tested by : Ostrander, J.R. [3.4]
Test identification: Test 15

Beam size : W12 × 27
Column size : W8 × 40
End-plate : 13 × 7.5 × 0.25 in.

Major parameters :	Lb=7.000in	Ct=0.500in	P=0.000in
	Li=7.000in	Cc=0.500in	G=4.000in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners : Column web stiffners opposite both beam flanges (7 × 4 × 0.250 in.)
Beam fastening : Weld
Column fastening : 4 × 3/4 in. diameter A325 bolts
Hole type : 13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	58.0	78.8
Beam web:	64.2	82.0
Column flange:	42.0	71.6
Column web:	48.6	75.0
End-plate:	42.4	71.1

Average pretension force of bolts :40 kips (0.0123 in. elongation)

Failure moment: 870 kips.in
Failure mode : Bolt failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B15 : Moment-Rotation Data : Ostrander - Test 15

Moment (Kips.in)	Rotation (radians)
0.	0.0000
94	0.0004
188	0.0010
282	0.0014
376	0.0025
470	0.0038
517	0.0046
564	0.0055
611	0.0068
658	0.0086
703	0.0108
750	0.0136
795	0.0175
815	0.0194
846	0.0242

Table B16 : Moment-Rotation Data : Ostrander - Test 16
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 16

Beam size :	W12 × 27
Column size :	W8 × 40
End-plate :	13 × 7.5 × 0.625 in.

Major parameters :	Lb=7.000in	Ct=0.500in	P=0.000in
	Li=7.000in	Cc=0.500in	G=4.000in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners :	Column web stiffners opposite both beam flanges (7 × 4 × 0.250 in.)
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	58.0	78.8
Beam web:	64.2	82.0
Column flange:	42.0	71.6
Column web:	48.6	75.0
End-plate:	40.9	75.3

Average pretension force of bolts :41.7 kips (0.0200 in. elongation)

Failure moment:	1000 kips.in
Failure mode :	Bolt failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B16 : Moment-Rotation Data : Ostrander - Test 16

Moment (Kips.in)	Rotation (radians)
0.	0.0000
94	0.0003
188	0.0006
282	0.0009
376	0.0012
470	0.0019
517	0.0023
564	0.0028
611	0.0034
658	0.0041
702	0.0051
752	0.0065
794	0.0082
840	0.0107
891	0.0143
933	0.0185
975	0.0246
1000	0.0296

Table B17 : Moment-Rotation Data : Ostrander - Test 17
Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 17

Beam size :	W12 × 27
Column size :	W8 × 24
End-plate :	13 × 7.5 × 0.625 in.

Major parameters :	Lb=7.000in	Ct=0.500in	P=0.000in
	Li=7.000in	Cc=0.500in	G=4.000in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	58.0	78.8
Beam web:	64.2	82.0
Column flange:	-	-
Column web:	-	-
End-plate:	62.9	109.1

Average pretension force of bolts :41.7 kips (0.0190 in. elongation)

Failure moment: 578 kips.in
Failure mode : Bolt failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B17 : Moment-Rotation Data : Ostrander - Test 17

Moment (Kips.in)	Rotation (radians)
0.	0.0000
94	0.0008
188	0.0020
282	0.0039
376	0.0084
423	0.0128
465	0.0196
505	0.0310
540	0.0433
569	0.0613
578	0.0746

Table B18 : Moment-Rotation Data : Ostrander - Test 18 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 18

Beam size :	W12 × 27
Column size :	W8 × 24
End-plate :	13 × 7.5 × 0.25 in.

Major parameters :	Lb=7.000in	Ct=0.500in	P=0.000in
	Li=7.000in	Cc=0.500in	G=4.000in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	58.0	78.8
Beam web:	64.2	82.0
Column flange:	-	-
Column web:	-	-
End-plate:	42.4	71.1

Average pretension force of bolts :40.4 kips (0.0122 in. elongation)

Failure moment: 680 kips.in
Failure mode : Bolt failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B18 : Moment-Rotation Data : Ostrander - Test 18

Moment (Kips.in)	Rotation (radians)
0.	0.0000
94	0.0013
188	0.0028
282	0.0047
376	0.0080
465	0.0133
516	0.0181
564	0.0258
606	0.0362
651	0.0476
675	0.0672
662	0.0792

Table B19 : Moment-Rotation Data : Ostrander - Test 19
Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 19

Beam size :	W12 × 27
Column size :	W8 × 24
End-plate :	13 × 7.5 × 0.625 in.

Major parameters :	Lb=7.000in	Ct=0.500in	P=0.000in
	Li=7.000in	Cc=0.500in	G=4.000in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	58.0	78.8
Beam web:	64.2	82.0
Column flange:	-	-
Column web:	-	-
End-plate:	40.9	73.1

Average pretension force of bolts :41.4 kips (0.0183 in. elongation)

Failure moment: 641 kips.in
Failure mode : Bolt failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B19 : Moment-Rotation Data : Ostrander - Test 19

Moment (Kips.in)	Rotation (radians)
0.	0.0000
47	0.0004
94	0.0009
141	0.0014
188	0.0020
235	0.0027
282	0.0036
329	0.0047
376	0.0061
420	0.0081
470	0.0109
517	0.0147
562	0.0212
592	0.0309
627	0.0441

Table B20 : Moment-Rotation Data : Ostrander - Test 20
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 20

Beam size :	W12 × 27
Column size :	W8 × 24
End-plate :	13 × 7.5 × 0.375 in.

Major parameters :	Lb=7.000in	Ct=0.500in	P=0.000in
	Li=7.000in	Cc=0.500in	G=4.000in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners :	Column web stiffeners opposite both beam flanges (7.125 × 3 × 0.25 in.)
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	58.0	78.8
Beam web:	64.2	82.0
Column flange:	-	-
Column web:	62.9	71.1
End-plate:	40.9	73.1

Average pretension force of bolts : 39.1 kips (0.0098 in. elongation)

Failure moment:	582 kips.in
Failure mode :	Bolt failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B20 : Moment-Rotation Data : Ostrander - Test 20

Moment (Kips.in)	Rotation (radians)
0.	0.0000
94	0.0011
188	0.0021
282	0.0037
376	0.0070
423	0.0102
470	0.0155
510	0.0242
555	0.0364
582	0.0473
608	0.0695

Table B21 : Moment-Rotation Data : Ostrander - Test 21
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 21

Beam size :	W12 × 27
Column size :	W8 × 24
End-plate :	13 × 7.5 × 0.25 in.

Major parameters :	Lb=7.000in	Ct=0.500in	P=0.000in
	Li=7.000in	Cc=0.500in	G=4.000in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners :	Column web stiffeners opposite both beam flanges (7.125 × 3 × 0.25 in.)
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	58.0	78.8
Beam web:	64.2	82.0
Column flange:	-	-
Column web:	42.4	71.1
End-plate:	41.4	57.0

Average pretension force of bolts : 40.6 kips (0.0133 in. elongation)

Failure moment: 810 kips.in
Failure mode : Bolt failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B21 : Moment-Rotation Data : Ostrander - Test 21

Moment (Kips.in)	Rotation (radians)
0.	0.0000
94	0.0010
188	0.0021
282	0.0037
376	0.0057
470	0.0091
513	0.0115
565	0.0150
610	0.0200
655	0.0256
698	0.0328
725	0.0395
738	0.0426
754	0.0465
785	0.0558
808	0.0660

Table B22 : Moment-Rotation Data : Ostrander - Test 22
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 22

Beam size :	W12 × 27
Column size :	W8 × 24
End-plate :	13 × 7.5 × 0.625 in.

Stiffeners :	Column web stiffeners opposite both beam flanges (7.125 × 3 × 0.25 in.)
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	58.0	78.8
Beam web:	64.2	82.0
Column flange:	-	-
Column web:	42.7	71.2
End-plate:	41.4	57.0

Average pretension force of bolts : 39.5 kips (0.0103 in. elongation)

Failure moment:	760 kips.in
Failure mode :	Bolt failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B22 : Moment-Rotation Data : Ostrander - Test 22

Moment (Kips.in)	Rotation (radians)
0.	0.0000
94	0.0005
188	0.0009
282	0.0016
376	0.0028
470	0.0053
511	0.0072
564	0.0101
604	0.0135
639	0.0187
698	0.0286
738	0.0362
756	0.0415

Table B23 : Moment-Rotation Data : Ostrander - Test 23 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 23

Beam size :	W12 × 27
Column size :	W8 × 48
End-plate :	13 × 7.5 × 0.625 in.

Major parameters :	Lb=7.000in	Ct=0.500in	P=0.000in
	Li=7.000in	Cc=0.500in	G=4.000in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	58.0	78.8
Beam web:	64.2	82.0
Column flange:	38.4	65.7
Column web:	43.6	65.7
End-plate:	40.9	73.1

Average pretension force of bolts : 41.7 kips (0.0196 in. elongation)

Failure moment: 910 kips.in
Failure mode : Bolt failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [ ]

Table B23 : Moment-Rotation Data : Ostrander - Test 23

Moment (Kips.in)	Rotation (radians)
0.	0.0000
94	0.0005
188	0.0010
282	0.0018
376	0.0026
423	0.0030
470	0.0035
517	0.0041
564	0.0048
609	0.0057
660	0.0070
721	0.0093
795	0.0132
840	0.0171
893	0.0238

Table B24 : Moment-Rotation Data : Ostrander - Test 24
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Ostrander, J.R. [3.4]
Test identification:	Test 24

Beam size :	W12 × 27
Column size :	W8 × 48
End-plate :	13 × 7.5 × 0.625 in.

Major parameters :	Lb=7.000in	Ct=0.500in	P=0.000in
	Li=7.000in	Cc=0.500in	G=4.000in
	E1t=2.500in	E2t=0.000in	
	E1c=2.500in	E2c=0.000in	

Stiffeners :	Column web stiffeners opposite both beam flanges (6.75 × 4 × 0.25 in.)
Beam fastening :	Weld
Column fastening :	4 × 3/4 in. diameter A325 bolts
Hole type :	13/16 in. column holes drilled and end-plate punched

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	58.0	78.8
Beam web:	64.2	82.0
Column flange:	38.4	65.7
Column web:	43.6	65.7
End-plate:	42.7	71.2

Average pretension force of bolts : 40 kips (0.0115 in. elongation)

Failure moment: 978 kips.in
Failure mode : Bolt failure

Remarks: 1) Moment-rotation data provided in numerical form in reference [3.4]

Table B24 : Moment-Rotation Data : Ostrander - Test 24

Moment (Kips.in)	Rotation (radians)
0.	0.0000
94	0.0003
188	0.0005
282	0.0008
376	0.0012
470	0.0017
564	0.0024
658	0.0034
747	0.0052
795	0.0064
839	0.0081
890	0.0103
928	0.0131
950	0.0146
963	0.0167
978	0.0182
955	0.0199

Table B25 : Moment-Rotation Data : Zoetemeijer - Test 2
Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P. [3.11]
Test identification:	Test 2

Beam size :	IPE 300
Column size :	HE 200A
End-plate :	325 × 200 × 32 mm

Major parameters :	Lb=210mm	Ct=0	P=60mm
	Li=150mm	Cc= -	G=100mm
	E1t=35mm	E2t=95mm	
	E1c+Cc=80mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	6 × M20 grade 10.9 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	264.5	451.0
Beam web:	340.0	507.5
Column flange:	301.0	460.5
Column web:	262.0	440.0
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: (Max. recorded moment 100 kNm)
Failure mode : Web buckling observed after testing

Remarks: 1) Moment-rotation curve supplied privately
2) End-plate extends below bottom beam flange

Table B25 : Moment-Rotation Data : Zoetemeijer - Test 2

Moment (kNm)	Rotation (radians)
0.00	0.0000
4.84	0.00032
10.50	0.00053
15.30	0.00073
19.35	0.00106
24.20	0.00130
29.85	0.00159
33.87	0.00188
40.30	0.00260
44.35	0.00391
50.00	0.00578
54.84	0.00798
61.30	0.01400
69.35	0.01978
75.00	0.02377
79.84	0.02645
85.50	0.03223
89.50	0.03858
94.35	0.04656

Table B26 : Moment-Rotation Data : Zoetemeijer - Test 22 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P. [3.11]
Test identification:	Test 22

Beam size :	IPE 400
Column size :	HE 300A
End-plate :	425 × 270 × 32 mm

Major parameters :	Lb=295mm	Ct=0mm	P=65mm
	Li=230mm	Cc= -	G=140mm
	E1t=40mm	E2t=105mm	
	E1c+Cc=90mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	6 × M22 grade 10.9 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	317.5	417.0
Beam web:	356.0	446.5
Column flange:	257.0	399.0
Column web:	265.5	428.0
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: (Max. recorded moment 206 kNm)
Failure mode : Web buckling observed after testing

Remarks: 1) Moment-rotation curve supplied privately
2) 350 × 16 mm backing plate

Table B26 : Moment-Rotation Data : Zoetemeijer - Test 22

Moment (kNm)	Rotation (radians)
0.00	0.0000
10.810	0.0002
25.060	0.0004
39.315	0.00065
55.530	0.0010
71.250	0.0012
86.600	0.0015
100.00	0.0018
116.480	0.00225
134.640	0.0030
148.500	0.00404
163.640	0.0063
170.520	0.0078
177.900	0.00945
185.250	0.01165
193.610	0.01710
200.000	0.02000
205.400	0.02580

Table B27 : Moment-Rotation Data : Zoetemeijer - Test M5A Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P. [3.11]
Test identification:	Test M5A

Beam size :	IPE 300
Column size :	HE 240A
End-plate :	320 × 180 × 25 mm

Major parameters :	Lb= -	Ct= -	P= -
	Li= -	Cc= -	G= -
	E1t= -	E2t= -	
	E1c= -	E2c= -	

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	6 × M22 grade 10.9 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	264.5	451.0
Beam web:	340.0	507.5
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment:	160 kNm
Failure mode :	Column web buckling observed

Remarks:
1) Moment-rotation curve provided in reference[3.11]
2) 260 × 95 × 15 mm backing plate and end-plate extends below bottom beam flange by 20 mm

Table B27 : Moment-Rotation Data : Zoetemeijer - Test M5A

Moment (kNm)	Rotation (radians)
0.00	0.0000
37.00	0.00108
68.00	0.00175
85.85	0.00345
104.30	0.00615
112.30	0.00845
126.15	0.01290
138.45	0.02000
146.15	0.02460
152.90	0.03385
153.50	0.04000
153.40	0.05730
135.40	0.07692
128.60	0.07923
124.00	0.08538

Table B28 : Moment-Rotation Data : Zoetemeijer - Test M5B Connection Type : Flush end-plate connection to unstiffened column

Tested by : Zoetemeijer, P. [3.11]
Test identification: Test M5B

Beam size : IPE 300
Column size : HE 240A
End-plate : 350 × 180 × 25 mm

Major parameters :			
	Lb= -	Ct= -	P= -
	Li= -	Cc= -	G= -
	E1t= -	E2t= -	
	E1c= -	E2c= -	

Stiffeners : No
Beam fastening : Weld
Column fastening : 6 × M22 grade 10.9 bolts
Hole type : -

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	264.5	451.0
Beam web:	340.0	507.5
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 155 kNm
Failure mode : Column web buckling observed

Remarks: 1) Moment-rotation curve provided in reference[3.11]
2) 260 × 95 × 15 mm backing plate and end-plate extends below bottom beam flange by 50 mm

Table B28 : Moment-Rotation Data : Zoetemeijer - Test M5B

Moment (kNm)	Rotation (radians)
0.00	0.0000
37.00	0.00108
68.00	0.00175
85.85	0.00195
110.75	0.00460
129.25	0.00770
138.45	0.01000
150.75	0.01460
160.00	0.01885
156.80	0.03425
147.70	0.04000
149.25	0.04345
140.00	0.05575

Table B29 : Moment-Rotation Data : Zoetemeijer - Test M5C Connection Type : Flush end-plate connection to unstiffened column

Tested by : Zoetemeijer, P. [3.11]
Test identification: Test M5C

Beam size : IPE 300
Column size : HE 240A
End-plate : 350 × 180 × 25 mm

Major parameters :	Lb= -	Ct= -	P= -
	Li= -	Cc= -	G= -
	E1t= -	E2t= -	
	E1c= -	E2c= -	

Stiffeners : No
Beam fastening : Weld
Column fastening : 6 × M22 grade 10.9 bolts
Hole type : -

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	264.5	451.0
Beam web:	340.0	507.5
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 160 kNm
Failure mode : Column web buckling observed

Remarks: 1) Moment-rotation curve provided in reference[3.11]
2) 260 × 95 × 15 mm backing plate and end-plate extends below bottom beam flange by 50 mm

Table B29 : Moment-Rotation Data : Zoetemeijer - Test M5C

Moment (kNm)	Rotation (radians)
0.00	0.0000
30.75	0.00055
55.40	0.00154
86.15	0.00345
104.00	0.00615
120.00	0.00915
143.00	0.01885
149.25	0.02230
160.00	0.03077
169.25	0.04200
160.00	0.04845
148.00	0.05460

Table B30 : Moment-Rotation Data : Zoetemeijer - Test M5D Connection Type : Flush end-plate connection to unstiffened column

Tested by : Zoetemeijer, P. [3.11]
Test identification: Test M5D

Beam size : IPE 300
Column size : HE 240A
End-plate : 350 × 180 × 25 mm

Major parameters :	Lb= -	Ct= -	P= -
	Li= -	Cc= -	G=4.000in
	E1t= -	E2t= -	
	E1c= -	E2c= -	

Stiffeners : No
Beam fastening : Weld
Column fastening : 6 × M22 grade 10.9 bolts
Hole type : -

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	264.5	451.0
Beam web:	340.0	507.5
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 172 kNm
Failure mode : Column web buckling observed

Remarks: 1) Moment-rotation curve provided in reference[3.11]
2) 260 × 95 × 15 mm backing plate and end-plate extends below bottom beam flange by 50 mm

Table B30 : Moment-Rotation Data : Zoetemeijer - Test M5D

Moment (kNm)	Rotation (radians)
0.00	0.0000
30.75	0.00055
55.40	0.00154
86.15	0.00345
104.60	0.00485
126.15	0.00885
152.30	0.01670
160.00	0.02845
170.80	0.04000
175.40	0.04730
173.85	0.06000
173.00	0.06540
166.15	0.08305

Table B31 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 7
Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P and Kolstein, M.H. [3.12]
Test identification:	Test 7

Beam size :	IPE 400
Column size :	HE 240A
End-plate :	400 × 180 × 25 mm

Major parameters :	Lb=290mm	Ct=0mm	P=70mm
	Li=220mm	Cc=0mm	G=70mm
	E1t=50mm	E2t=120mm	
	E1c=50mm	E2c= -	

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	4 × M24 × 75 grade 8.8 bolts on tension 2 × M24 × 70 grade 8.8 bolts on compression side
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	306	432
Beam web:	336	462
Column flange:	291	419
Column web:	300	420
End-plate:	294	444

Average pretension force of bolts : Left beam 99 kN
Right beam 133.1 kN
Average 116 kN

Failure moment: 145.2 kNm
Failure mode : Buckling of column web

Remarks: 1) Moment-rotation curve provided in reference[3.12]
2) 214 × 95 × 15 mm backing plate on tension side

Table B31 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 7

Moment (kNm)	Rotation (radians)
0.00	0.0000
14.350	0.00035
28.710	0.00067
41.750	0.00087
51.900	0.00111
64.050	0.00156
75.090	0.00200
86.135	0.00223
99.390	0.00267
108.220	0.00290
120.000	0.00365
132.515	0.00600
140.000	0.00800
145.325	0.00980

Table B32 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 8
Connection Type : Flush end-plate connection to unstiffened column

Tested by : Zoetemeijer, P and Kolstein, M.H. [3.12]
Test identification: Test 8

Beam size : IPE 400
Column size : HE 240A
End-plate : 450 × 180 × 25 mm

Major parameters :	Lb=290mm	Ct=0mm	P=70mm
	Li=220mm	Cc=0mm	G=70mm
	E1t=50mm	E2t=120mm	
	E1c=50mm	E2c= -	

Stiffeners : No
Beam fastening : Weld
Column fastening : 4 × M24 × 75 grade 8.8 bolts on tension side
2 × M24 × 70 grade 8.8 bolts on compression side
Hole type : -

Average pretension force of bolts : Left beam 127.6 kN
Right beam 126.5 kN
Average 127 kN

Failure moment: 168.6 kNm
Failure mode : Buckling of culumn web

Remarks: 1) Moment-rotation curve provided in reference[3.12]
2) 214 × 95 × 15 mm backing plate on the tension side
and end-plate extends by 50mm below bottom beam flange

Table B32 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 8

Moment (kNm)	Rotation (radians)
0.00	0.0000
27.600	0.00030
37.775	0.00055
48.590	0.00085
61.840	0.00110
71.780	0.00140
83.925	0.00196
99.390	0.00200
108.220	0.00228
120.000	0.00267
133.620	0.00379
147.535	0.00557
157.915	0.00752
167.850	0.00958

Table B33 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 9
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Zoetemeijer, P and Kolstein, M.H. [3.12]
Test identification:	Test 9

Beam size :	IPE 400
Column size :	HE 240A
End-plate :	400 × 180 × 25 mm

Major parameters :	Lb=290mm	Ct=0mm	P=70mm
	Li=220mm	Cc=0mm	G=70mm
	E1t=50mm	E2t=120mm	
	E1c=50mm	E2c= -	

Stiffeners :	Column web stiffeners on compression side only (10mm thick)
Beam fastening :	Weld
Column fastening :	6 × M24 × 70 grade 8.8 bolts on tension side
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	306	432
Beam web:	336	462
Column flange:	291	419
Column web:	300	420
End-plate:	294	444

Average pretension force of bolts : Left beam 68 kN
Right beam 62 kN
Average 65 kN

Failure moment: 216 kNm
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation curve provided in reference[3.12]

Table B33 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 9

Moment (kNm)	Rotation (radians)
0.00	0.0000
20.000	0.00020
40.000	0.00044
60.000	0.00083
76.750	0.00115
86.690	0.00200
99.390	0.00400
110.000	0.00668
114.850	0.00800
121.475	0.01000
126.330	0.01200
131.410	0.01400
135.000	0.01600
140.000	0.01800
145.550	0.02000
150.000	0.02200
154.600	0.02400
158.800	0.02600
161.000	0.02800
164.550	0.03000

Table B34 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 10
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Zoetemeijer, P and Kolstein, M.H. [3.12]
Test identification:	Test 10

Beam size :	IPE 400
Column size :	HE 240A
End-plate :	400 × 180 × 25 mm

Major parameters :	Lb=290mm	Ct=0mm	P=70mm
	Li=220mm	Cc=0mm	G=70mm
	E1t=50mm	E2t=120mm	
	E1c=50mm	E2c= -	

Stiffeners :	Column web stiffeners on compression side only (10mm thick)
Beam fastening :	Weld
Column fastening :	4 × M24 × 75 grade 8.8 bolts on tension side
	2 × M24 × 70 grade 8.8 bolts on compression side
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	306	432
Beam web:	336	462
Column flange:	291	419
Column web:	300	420
End-plate:	294	444

Average pretension force of bolts : Left beam 55.6 kN
Right beam 74.4 kN
Average 65 kN

Failure moment: 276 kNm
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation curve provided in reference[3.12]
2) 214 × 95 × 15 mm backing plate on tension side

Table B34 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 10

Moment (kNm)	Rotation (radians)
0.00	0.0000
8.850	0.00022
25.400	0.000390
40.000	0.00050
600.000	0.00072
72.885	0.00114
87.240	0.00178
97.180	0.00245
106.000	0.00312
120.000	0.00380
140.000	0.00512
150.180	0.00600
160.000	0.00745
176.580	0.00913
182.760	0.01000
188.840	0.01155
194.355	0.01290
201.000	0.01400
207.400	0.01560
210.920	0.01720
220.000	0.0200

Table B35 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 11
Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P and Kolstein, M.H. [3.12]
Test identification:	Test 11

Beam size :	IPE 400
Column size :	HE 240A
End-plate :	450 × 180 × 25 mm

Major parameters :	Lb=290mm	Ct=0mm	P=70mm
	Li=220mm	Cc=0mm	G=70mm
	E1t=50mm	E2t=120mm	
	E1c=50mm	E2c= -	

Stiffeners :	-
Beam fastening :	Weld
Column fastening :	6 × M24 × 70 grade 8.8 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	306	432
Beam web:	336	462
Column flange:	291	419
Column web:	300	420
End-plate:	294	444

Average pretension force of bolts : Left beam 85.6 kN
Right beam 85.0 kN
Average 85 kN

Failure moment: 198 kNm
Failure mode : Buckling of column web

Remarks:
1) Moment-rotation curve provided in reference[3.12]
2) End-plate extends below beam bottom flanges

Table B35 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 11

Moment (kNm)	Rotation (radians)
0.00	0.0000
20.000	0.00022
36.440	0.00044
50.000	0.00067
60.000	0.00078
72.550	0.00111
80.000	0.00156
88.350	0.00200
104.35	0.00400
114.850	0.00600
123.680	0.00800
130.305	0.01000
136.930	0.0120
143.340	0.014000
148.000	0.01600
153.500	0.01800
156.810	0.02000
160.125	0.02200
165.090	0.02400
172.270	0.02955
180.000	0.03200
181.110	0.03676
187.730	0.03676
193.250	0.04180

Table B36 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 12
Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P and Kolstein, M.H. [3.12]
Test identification:	Test 12

Beam size :	IPE 400
Column size :	HE 240A
End-plate :	400 × 180 × 25 mm

Major parameters :	Lb=290mm	Ct=0mm	P=70mm
	Li=220mm	Cc=0mm	G=70mm
	E1t=50mm	E2t=120mm	
	E1c=50mm	E2c= -	

Stiffeners :	-
Beam fastening :	Weld
Column fastening :	6 × M24 × 70 grade 8.8 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	306	432
Beam web:	336	462
Column flange:	291	419
Column web:	300	420
End-plate:	294	444

Average pretension force of bolts : Left beam 65 kN
Right beam 92 kN
Average 78.5 kN

Failure moment: 168 kNm
Failure mode : Buckling of column web

Remarks: 1) Moment-rotation curve provided in reference[3.12]

Table B36 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 12

Moment (kNm)	Rotation (radians)
0.000	0.00000
23.750	0.00027
34.700	0.00041
47.500	0.00068
58.500	0.00091
73.000	0.00142
82.200	0.00220
96.800	0.00444
108.700	0.00756
115.000	0.01026
120.000	0.01246
138.800	0.02145
160.000	0.03602
164.400	0.04000

Table B37 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 13
Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P and Kolstein, M.H. [3.12]
Test identification:	Test 13

Beam size :	IPE 300
Column size :	HE 240A
End-plate :	300 × 150 × 25 mm

Major parameters :	Lb=210mm	Ct=0mm	P=70mm
	Li=140mm	Cc=0mm	G=70mm
	E1t=45mm	E2t=115mm	
	E1c=45mm	E2c=0mm	

Stiffeners :	-
Beam fastening :	Weld
Column fastening :	4 × M24 × 75 grade 8.8 bolts on tension side
	2 × 24 × 70 grade 8.8 bolts on compression
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	311	466
Beam web:	358	495
Column flange:	291	419
Column web:	300	420
End-plate:	294	444

Average pretension force of bolts : Left beam 45.5 kN
Right beam 61.7 kN
Average 54.0 kN

Failure moment: 120 kNm
Failure mode : Buckling of column web

Remarks: 1) Moment-rotation curve provided in reference[3.12]
2) 214 × 95 × 15 mm backing plate on tension side, end-plate extends by 50 mm below bottom beam flange.

Table B37 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 13

Moment (kNm)	Rotation (radians)
0.00	0.0000
13.250	0.00033
24.295	0.00067
36.440	0.00156
48.590	0.00312
60.000	0.00470
73.435	0.00557
83.930	0.01070
98.280	0.01570
103.800	0.01880
110.430	0.02484
115.400	0.03175

Table B38 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 14
Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P and Kolstein, M.H. [3.12]
Test identification:	Test 14

Beam size :	IPE 300
Column size :	HE 240A
End-plate :	350 × 150 × 25 mm

Major parameters :	Lb=210mm	Ct=0mm	P=70mm
	Li=140mm	Cc=0mm	G=70mm
	E1t=45mm	E2t=115mm	
	E1c=45mm	E2c=0mm	

Stiffeners :	-
Beam fastening :	Weld
Column fastening :	4 × M24 × 75 grade 8.8 bolts on tension side 2 × 24 × 70 grade 8.8 bolts on compression
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	311	466
Beam web:	358	495
Column flange:	291	419
Column web:	300	420
End-plate:	294	444

Average pretension force of bolts : Left beam 65.5 kN
Right beam 59.5 kN
Average 62.5 kN

Failure moment: 156 kNm
Failure mode : Buckling of column web

Remarks: 1) Moment-rotation curve provided in reference[3.12]
2) 214 × 95 × 15 mm backing plate on tension side, end-plate extends by 50 mm below bottom beam flange.

Table B38 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 14

Moment (kNm)	Rotation (radians)
0.00	0.0000
14.360	0.00045
27.055	0.00072
40.000	0.00111
50.800	0.00200
60.750	0.00301
76.200	0.00480
85.030	0.00615
97.730	0.00825
103.800	0.00958
111.530	0.01142
117.056	0.01314
121.475	0.01582
125.890	0.01782
135.850	0.02306
140.000	0.02740
145.770	0.03308
153.500	0.04088

Table B39 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 15
Connection Type : Flush end-plate connection to stiffened column

Tested by : Zoetemeijer, P and Kolstein, M.H. [3.12]
Test identification: Test 15

Beam size : IPE 300
Column size : HE 240A
End-plate : 300 × 150 × 25 mm

Major parameters :	Lb=210mm	Ct=0mm	P=70mm
	Li=140mm	Cc=0mm	G=70mm
	E1t=45mm	E2t=115mm	
	E1c=45mm	E2c=0mm	

Stiffeners : Column web stiffeners on compression side only 10 mm thick
Beam fastening : Weld
Column fastening : 4 × M24 × 75 grade 8.8 bolts
Hole type : -

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	311	466
Beam web:	358	495
Column flange:	291	419
Column web:	300	420
End-plate:	294	444

Average pretension force of bolts : Left beam 63 kN
Right beam 78.4 kN
Average 70.5 kN

Failure moment: 129 kNm
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation curve provided in reference[ ]

Table B39 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 15

Moment (kNm)	Rotation (radians)
0.00	0.0000
14.350	0.00033
20.000	0.00047
36.440	0.00106
48.590	0.00200
60.730	0.00400
66.260	0.00600
70.890	0.00800
73.990	0.01000
77.300	0.01200
80.800	0.01400
83.920	0.01600
87.800	0.02000
92.760	0.02400
96.000	0.02800
100.000	0.03200
103.800	0.03600
106.000	0.04000
110.430	0.04380

Table B40 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 16
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Zoetemeijer, P and Kolstein, M.H. [3.12]
Test identification:	Test 16

Beam size :	IPE 300
Column size :	HE 240A
End-plate :	300 × 150 × 25 mm

Major parameters :	Lb=210mm	Ct=0mm	P=70mm
	Li=140mm	Cc=0mm	G=70mm
	E1t=45mm	E2t=115mm	
	E1c=45mm	E2c=0mm	

Stiffeners :	Column web stiffeners on compression side only 10 mm thick
Beam fastening :	Weld
Column fastening :	4 × M24 × 75 grade 8.8 bolts on tension side
	2 × M24 × 70 grade 8.8 bolts on compression side
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	311	466
Beam web:	358	495
Column flange:	291	419
Column web:	300	420
End-plate:	294	444

Average pretension force of bolts : Left beam 79.4 kN
Right beam 68.3 kN
Average 74 kN

Failure moment: 168 kNm
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation curve provided in reference[3.12]
2) 214 × 95 × 15 mm backing plate on the tension side.

Table B40 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 16

Moment (kNm)	Rotation (radians)
0.00	0.0000
13.250	0.00022
26.500	0.00066
35.338	0.00089
49.700	0.00167
60.000	0.00267
71.780	0.00406
88.340	0.00600
95.520	0.00780
106.000	0.00958
109.300	0.01092
112.640	0.01200
121.470	0.01500
128.100	0.01738
136.600	0.02450
140.000	0.02996
147.425	0.03676
158.900	0.04300

Table B41 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 17
Connection Type : Flush end-plate connection to unstiffened column

Tested by : Zoetemeijer, P and Kolstein, M.II. [3.12]
Test identification: Test 17

Beam size : IPE 300
Column size : HE 240A
End-plate : 350 × 150 × 25 mm

Major parameters :	Lb=210mm	Ct=0mm	P=70mm
	Li=140mm	Cc=0mm	G=70mm
	E1t=45mm	E2t=115mm	
	E1c=45mm	E2c=0mm	

Stiffeners : No
Beam fastening : Weld
Column fastening : 6 × M24 × 70 grade 8.8 bolts
Hole type : -

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	311	466
Beam web:	358	495
Column flange:	291	419
Column web:	300	420
End-plate:	294	444

Average pretension force of bolts : Left beam 88.9 kN
Right beam 107.8 kN
Average 98 kN

Failure moment: 130.2 kNm
Failure mode : Buckling of column web

Remarks: 1) Moment-rotation curve provided in reference[3.12]
2) End-plate extends below bottom beam flanges by 50 mm.

Table B41 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 17

Moment (kNm)	Rotation (radians)
0.00	0.0000
13.250	0.00028
18.780	0.00083
49.700	0.00178
61.840	0.00312
70.675	0.00600
76.200	0.00800
80.060	0.01000
85.030	0.01400
88.345	0.01600
91.660	0.02000
100.000	0.02900
103.805	0.03400
110.430	0.04200

Table B42 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 18
Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P and Kolstein, M.H. [3.12]
Test identification:	Test 18

Beam size :	IPE 300
Column size :	HE 240A
End-plate :	300 × 150 × 25 mm

Major parameters :	Lb=210mm	Ct=0mm	P=70mm
	Li=140mm	Cc=0mm	G=70mm
	E1t=45mm	E2t=115mm	
	E1c=45mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	6 × M24 × 70 grade 8.8 bolts
Hole type :	-

Measured Material Properties:
All sections grade steel

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	311	466
Beam web:	358	495
Column flange:	291	419
Column web:	300	420
End-plate:	294	444

Average pretension force of bolts :	Left beam 127 kN
	Right beam 113 kN
	Average 120 kN

Failure moment:	95.4 kNm
Failure mode :	Buckling of column web

Remarks: 1) Moment-rotation curve provided in reference[3.12]

Table B42 : Moment-Rotation Data : Zoetemeijer and Kolstein - Test 18

Moment (kNm)	Rotation (radians)
0.000	0.00000
13.750	0.00041
27.500	0.00082
36.650	0.00128
57.750	0.00595
64.170	0.00980
71.500	0.01723
80.000	0.02800
84.350	0.04377

Table B43 : Moment-Rotation Data : Zoetemeijer - Test 16 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 16

Beam size :	HE 300A
Column size :	HE 450M
End-plate :	290 × 230 × 12 mm

Major parameters :	Lb=180mm	Ct=0mm	P=0mm
	Li=180mm	Cc=0mm	G=120mm
	E1t=55mm	E2t=0mm	
	E1c=55mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	10 mm welds on inside and outside of end-plate
Column fastening :	4 × M24 grade 8.8 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	294	-

Average pretension force of bolts : 1st row 48 kN
2nd row 40 kN

Failure moment:	Max recorded moment 124 kNm
Failure mode :	Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B43 : Moment-Rotation Data : Zoetemeijer - Test 16

Moment (kNm)	Rotation (radians)
0.00	0.0000
15.560	0.00040
23.340	0.00090
30.150	0.00160
37.000	0.00240
45.700	0.00350
63.210	0.00510
62.220	0.00830
70.020	0.01260
77.800	0.01400
80.700	0.02010
83.635	0.02260
87.525	0.02480
91.515	0.02720
93.360	0.02950
99.250	0.03450
102.110	0.03700
104.050	0.04150
105.000	0.04330
110.100	0.05120
114.800	0.06220
117.000	0.06520

Table B44 : Moment-Rotation Data : Zoetemeijer - Test 17 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 17

Beam size :	HE 300A
Column size :	HE 450M
End-plate :	290 × 230 × 16 mm

Major parameters :	Lb=180mm	Ct=0mm	P=0mm
	Li=180mm	Cc=0mm	G=120mm
	E1t=55mm	E2t=0mm	
	E1c=55mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	10 mm welds on inside and outside of end-plate
Column fastening :	4 × M22 grade 8.8 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	286	-

Average pretension force of bolts : 1st row 40 kN
2nd row 40 kN

Failure moment:	Max recorded moment 147 kNm
Failure mode :	Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B44 : Moment-Rotation Data : Zoetemeijer - Test 17

Moment (kNm)	Rotation (radians)
0.00	0.0000
15.560	0.00060
24.300	0.00100
33.065	0.00150
38.900	0.00240
46.680	0.00300
56.405	0.00370
64.185	0.00470
70.020	0.00600
77.800	0.00750
87.525	0.00970
95.305	0.01280
101.000	0.01610
109.890	0.02010
113.780	0.02170
116.700	0.02400
119.615	0.02640
122.535	0.02780
126.425	0.02950
130.315	0.03110
132.260	0.03230
134.205	0.03350
137.610	0.03550
141.000	0.03760
143.930	0.03860

Table B45 : Moment-Rotation Data : Zoetemeijer - Test 18 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 18

Beam size :	HE 300A
Column size :	HE 450M
End-plate :	290 × 230 × 12 mm

Major parameters :	Lb=140mm	Ct=0mm	P=0mm
	Li=140mm	Cc=0mm	G=120mm
	E1t=75mm	E2t=0mm	
	E1c=75mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	10 mm welds on inside and outside of end-plate
Column fastening :	4 × M24 grade 8.8 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	294	-

Average pretension force of bolts : 1st row 44 kN
2nd row 30 kN

Failure moment: 117 kNm
Failure mode : Weld

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B45 : Moment-Rotation Data : Zoetemeijer - Test 18

Moment (kNm)	Rotation (radians)
0.00	0.0000
16.535	0.00040
28.400	0.00200
45.705	0.00510
54.460	0.00810
62.240	0.01280
70.020	0.02080
76.825	0.03100
84.605	0.04210
91.415	0.06030
93.360	0.06030
95.305	0.06480

Table B46 : Moment-Rotation Data : Zoetemeijer - Test 19 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 19

Beam size :	HE 300A
Column size :	HE 450M
End-plate :	290 × 230 × 16 mm

Major parameters :	Lb=140mm	Ct=0mm	P=0mm
	Li=140mm	Cc=0mm	G=120mm
	E1t=75mm	E2t=0mm	
	E1c=75mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	10 mm welds on inside and outside of end-plate
Column fastening :	4 × M24 grade 8.8 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	286	-

Average pretension force of bolts : 1st row 36 kN
2nd row 32 kN

Failure moment: Max. reached moment 128 kNm
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B46 : Moment-Rotation Data : Zoetemeijer - Test 19

Moment (kNm)	Rotation (radians)
0.00	0.0000
14.600	0.00020
29.175	0.00220
37.000	0.00350
44.735	0.00470
52.520	0.00550
61.270	0.00750
70.020	0.00960
76.820	0.01160
84.605	0.01890
91.415	0.02560
100.105	0.03270
106.975	0.04310
110.865	0.04900
112.810	0.05550

Table B47 : Moment-Rotation Data : Zoetemeijer - Test 20 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 20

Beam size :	HE 300A
Column size :	HE 450M
End-plate :	290 × 230 × 12 mm

Major parameters :	Lb=180mm	Ct=0mm	P=0mm
	Li=180mm	Cc=0mm	G=230mm
	E1t=55mm	E2t=0mm	
	E1c=55mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	10 mm welds on inside and outside of end-plate
Column fastening :	4 × M24 grade 8.8 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	294	-

Average pretension force of bolts : 1st row 72 kN
2nd row 60 kN

Failure moment: Max. reached moment 71 kNm
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B47 : Moment-Rotation Data : Zoetemeijer - Test 20

Moment (kNm)	Rotation (radians)
0.00	0.0000
7.780	0.00060
15.075	0.00270
23.340	0.00510
31.120	0.00750
37.930	0.01150
46.680	0.01770
54.460	0.02900
62.240	0.04390
64.185	0.04920
70.020	0.06380

Table B48 : Moment-Rotation Data : Zoetemeijer - Test 21 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 21

Beam size :	HE 300A
Column size :	HE 450M
End-plate :	290 × 230 × 16 mm

Major parameters :	Lb=180mm	Ct=0mm	P=0mm
	Li=180mm	Cc=0mm	G=230mm
	E1t=55mm	E2t=0mm	
	E1c=55mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	6 mm welds on inside and outside of end-plate
Column fastening :	4 × M24 grade 8.8 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	286	-

Average pretension force of bolts : 1st row 40 kN
2nd row 52 kN

Failure moment:	Max. reached moment 93 kNm
Failure mode :	Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B48 : Moment-Rotation Data : Zoetemeijer - Test 21

Moment (kNm)	Rotation (radians)
0.00	0.0000
7.780	0.00030
15.560	0.00120
23.340	0.00250
31.120	0.00390
36.955	0.00570
44.735	0.00780
53.487	0.01120
62.240	0.01770
64.185	0.01890
60.075	0.02340
71.965	0.03680
73.910	0.03080
75.855	0.03620
79.745	0.04020
82.665	0.04490
85.580	0.05080
107.948	0.05470

Table B49 : Moment-Rotation Data : Zoetemeijer - Test 22
Connection Type : Flush end-plate connection to unstiffened column

Tested by : Zoetemeijer, P [3.13]
Test identification: Test 22

Beam size : HE 300A
Column size : HE 450M
End-plate : 290 × 230 × 12 mm

Major parameters :			
	Lb=140mm	Ct=0mm	P=0mm
	Li=140mm	Cc=0mm	G=230mm
	E1t=75mm	E2t=0mm	
	E1c=75mm	E2c=0mm	

Stiffeners : No
Beam fastening : 6mm welds on inside and outside of end-plate
Column fastening : 4 × M24 grade 8.8 bolts
Hole type : -

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	294	-

Average pretension force of bolts : 1st row 48 kN
2nd row 40 kN

Failure moment: Max. recorded moment 50 kNm
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B49 : Moment-Rotation Data : Zoetemeijer - Test 22

Moment (kNm)	Rotation (radians)
0.000	0.0000
7.780	0.0003
15.560	0.0018
21.395	0.0045
31.120	0.0106
33.065	0.0143
35.982	0.0205
38.900	0.0276
41.075	0.0360
45.225	0.0441
46.925	0.0526

Table B50 : Moment-Rotation Data : Zoetemeijer - Test 23 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 23

Beam size :	HE 300A
Column size :	HE 450M
End-plate :	290 × 230 × 16 mm

Major parameters :	Lb=140mm	Ct=0mm	P=0mm
	Li=140mm	Cc=0mm	G=230mm
	E1t=75mm	E2t=0mm	
	E1c=75mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	6mm welds on inside and outside of end-plate
Column fastening :	4 × M24 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	286	-

Average pretension force of bolts : 1st row 48 kN
2nd row 56 kN

Failure moment: Max. recorded moment 68 kNm
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B50 : Moment-Rotation Data : Zoetemeijer - Test 23

Moment (kNm)	Rotation (radians)
0.000	0.0000
15.075	0.0008
23.340	0.0027
31.120	0.0055
38.900	0.0095
41.817	0.0130
44.735	0.0170
48.625	0.0209
52.515	0.0252
54.946	0.0280
58.350	0.0350
60.780	0.0423
64.185	0.0502
68.075	0.0681

Table B51 : Moment-Rotation Data : Zoetemeijer - Test 24 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 24

Beam size :	HE 300A
Column size :	HE 450M
End-plate :	290 × 230 × 12 mm

Major parameters :	Lb=180mm	Ct=0mm	P=0mm
	Li=125mm	Cc=0mm	G=120mm
	E1t=55mm	E2t=110mm	
	E1c=55mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	6mm welds on inside and outside of end-plate
Column fastening :	6 × M24 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	294	-

Average pretension force of bolts : 1st row 48 kN
2nd row 40 kN
3rd row not measured

Failure moment:	Max. recorded moment 124 kNm
Failure mode :	Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B51 : Moment-Rotation Data : Zoetemeijer - Test 24

Moment (kNm)	Rotation (radians)
0.000	0.0000
14.587	0.0005
23.340	0.0012
31.120	0.0022
37.927	0.0032
44.735	0.0043
53.515	0.0056
62.240	0.0081
78.772	0.0154
81.690	0.0173
83.635	0.0189
88.497	0.0213
91.415	0.0234
93.360	0.0256
97.250	0.0280
99.195	0.0311
101.140	0.0337
105.030	0.0358
107.947	0.0398
110.865	0.0425
112.810	0.0469
115.241	0.0502

Table B52 : Moment-Rotation Data : Zoetemeijer - Test 25 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 25

Beam size :	HE 300A
Column size :	HE 450M
End-plate :	290 × 230 × 16 mm

Major parameters :	Lb=180mm	Ct=0mm	P=0mm
	Li=125mm	Cc=0mm	G=120mm
	E1t=55mm	E2t=110mm	
	E1c=55mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	6mm welds on inside and outside of end-plate
Column fastening :	6 × M24 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	286	-

Average pretension force of bolts : 1st row 40 kN
2nd row 40 kN
2rd row not measured

Failure moment:	Max. recorded moment 148 kNm
Failure mode :	Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B52 : Moment-Rotation Data : Zoetemeijer - Test 25

Moment (kNm)	Rotation (radians)
0.000	0.0000
6.809	0.0003
14.587	0.0004
21.395	0.0008
31.120	0.0016
36.955	0.0026
44.735	0.0037
53.487	0.0043
60.295	0.0050
68.075	0.0063
75.855	0.0075
84.607	0.0094
92.387	0.0119
100.167	0.0160
109.892	0.0217
112.810	0.0240
114.755	0.0260
119.617	0.0287
122.535	0.0308
125.452	0.0333
128.370	0.0356
132.260	0.0386
134.205	0.0413
138.095	0.0453
140.040	0.0486
141.985	0.0520
145.875	0.0571

Table B53 : Moment-Rotation Data : Zoetemeijer - Test 26 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 26

Beam size :	HE 300A
Column size :	HE 450M
End-plate :	290 × 230 × 12 mm

Major parameters :	Lb=180mm	Ct=0mm	P=0mm
	Li=180mm	Cc=0mm	G=120mm
	E1t=55mm	E2t=0mm	
	E1c=55mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	6mm welds on inside and outside of end-plate
Column fastening :	6 × M24 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	294	-

Average pretension force of bolts : 1st row (inner bolts) 48 kN
1st row (outer bolts) 36 kN

Failure moment: Max. recorded moment 130 kNm
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange
3) 4 Bolts in the tension row

Table B53 : Moment-Rotation Data : Zoetemeijer - Test 26

Moment (kNm)	Rotation (radians)
0.000	0.0000
8.170	0.0004
15.560	0.0006
21.395	0.0014
30.147	0.0022
38.900	0.0032
45.707	0.0047
53.480	0.0060
61.276	0.0081
70.020	0.0106
76.830	0.0134
85.580	0.0175
93.360	0.0221
95.790	0.0236
101.626	0.0280
107.950	0.0327
110.865	0.0358
113.296	0.0384
114.755	0.0410
119.617	0.0445
123.507	0.0467
125.452	0.0504
128.370	0.0532

Table B54 : Moment-Rotation Data : Zoetemeijer - Test 27 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 27

Beam size :	HE 300A
Column size :	HE 450M
End-plate :	290 × 230 × 16 mm

Major parameters :	Lb=180mm	Ct=0mm	P=0mm
	Li=180mm	Cc=0mm	G=120mm
	E1t=55mm	E2t=0mm	
	E1c=55mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	6mm welds on inside and outside of end-plate
Column fastening :	6 × M24 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	286	-

Average pretension force of bolts : 1st row (inner bolts) 44 kN
1st row (outer bolts) 35 kN
2nd row 26kN

Failure moment: Max. recorded moment 160 kNm
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange
3) 4 Bolts in the tension row

Table B54 : Moment-Rotation Data : Zoetemeijer - Test 27

Moment (kNm)	Rotation (radians)
0.000	0.0000
7.800	0.0002
14.875	0.0005
21.395	0.0010
30.147	0.0016
37.928	0.0024
44.735	0.0032
54.460	0.0037
62.240	0.0044
68.560	0.0053
77.800	0.0063
85.580	0.0075
94.335	0.0087
101.140	0.0107
108.920	0.0132
116.700	0.0165
124.480	0.0201
130.315	0.0230
133.235	0.0252
136.150	0.0251
140.040	0.0271
143.443	0.0303
144.902	0.0329
147.820	0.0350
151.710	0.0378
155.600	0.0410
158.031	0.0429
162.407	0.0453

Table B55 : Moment-Rotation Data : Zoetemeijer - Test 28 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 28

Beam size :	HE 300A
Column size :	HE 450M
End-plate :	290 × 230 × 12 mm

Major parameters :	Lb=140mm	Ct=0mm	P=0mm
	Li=140mm	Cc=0mm	G=120mm
	E1t=75mm	E2t=0mm	
	E1c=75mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	6mm welds on inside and outside of end-plate
Column fastening :	6 × M24 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	294	-

Average pretension force of bolts : 1st row (inner bolts) 32 kN
1st row (outer bolts) 28 kN
2nd row 28 kN

Failure moment:	Max. recorded moment 90 kNm
Failure mode :	Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange
3) 4 bolts in the tension row

Table B55 : Moment-Rotation Data : Zoetemeijer - Test 28

Moment (kNm)	Rotation (radians)
0.000	0.0000
14.587	0.0004
21.395	0.0008
29.660	0.0024
36.955	0.0043
43.276	0.0066
53.000	0.0110
62.240	0.0169
68.075	0.0244
71.965	0.0312
75.855	0.0331
79.500	0.0336
81.690	0.0410
84.120	0.0469
89.180	0.0498

Table B56 : Moment-Rotation Data : Zoetemeijer - Test 29 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 29

Beam size :	HE 300A
Column size :	HE 450M
End-plate :	290 × 230 × 12 mm

Major parameters :	Lb=140mm	Ct=0mm	P=0mm
	Li=140mm	Cc=0mm	G=120mm
	E1t=75mm	E2t=0mm	
	E1c=75mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	6mm welds on inside and outside of end-plate
Column fastening :	6 × M24 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	294	-

Average pretension force of bolts : 1st row (inner bolts) 24 kN
1st row (outer bolts) 34 kN
2nd row 36 kN

Failure moment: Max. recorded moment 90 kNm
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange
3) 4 Bolts in the tension row

Table B56 : Moment-Rotation Data : Zoetemeijer - Test 29

Moment (kNm)	Rotation (radians)
0.000	0.0000
14.587	0.0004
23.340	0.0006
31.120	0.0014
38.600	0.0026
44.735	0.0039
52.515	0.0053
61.267	0.0070
68.075	0.0070
75.855	0.0125
84.607	0.0168
93.360	0.0222
95.305	0.0240
99.195	0.0272
102.000	0.0297
104.058	0.0331
107.947	0.0354
110.379	0.0390
112.810	0.0429
114.755	0.0465
119.617	0.0500
121.562	0.0551

Table B57 : Moment-Rotation Data : Zoetemeijer - Test 30 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 30

Beam size :	IPE 400
Column size :	HE 450M
End-plate :	400 × 180 × 12 mm

Major parameters :	Lb=290mm	Ct=0mm	P=70mm
	Li=220mm	Cc=0mm	G=100mm
	E1t=55mm	E2t=125mm	
	E1c=55mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	6mm welds on inside and outside of end-plate
Column fastening :	6 × M24 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	294	-

Average pretension force of bolts :	1st row 66 kN
	2nd row 54 kN
	3rd row 60 kN

Failure moment:	Max. recorded moment 195 kNm
Failure mode :	Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B57 : Moment-Rotation Data : Zoetemeijer - Test 30

Moment (kNm)	Rotation (radians)
0.000	0.0000
40.000	0.0002
69.475	0.0008
79.400	0.0012
102.225	0.0024
121.085	0.0040
131.010	0.0048
145.897	0.0199
160.785	0.0139
181.627	0.0221
189.567	0.0253
217.850	0.0426

Table B58 : Moment-Rotation Data : Zoetemeijer - Test 31
Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 31

Beam size :	IPE 400
Column size :	HE 450M
End-plate :	400 × 180 × 12 mm

Major parameters :	Lb=290mm	Ct=0mm	P=0mm
	Li=290mm	Cc=0mm	G=100mm
	E1t=55mm	E2t=0mm	
	E1c=55mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	6mm welds on inside and outside of end-plate
Column fastening :	4 × M24 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	294	-

Average pretension force of bolts : 1st row not measured
2nd row measured

Failure moment: Max. recorded moment 160 kNm
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B58 : Moment-Rotation Data : Zoetemeijer - Test 31

Moment (kNm)	Rotation (radians)
0.000	0.0000
16.875	0.0001
40.000	0.0005
65.505	0.0020
80.000	0.0036
92.300	0.0054
111.160	0.0115
131.010	0.0200
144.905	0.0264
153.340	0.0357
162.770	0.0468

Table B59 : Moment-Rotation Data : Zoetemeijer - Test 32 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 32

Beam size :	IPE 400
Column size :	HE 450M
End-plate :	400 × 180 × 12 mm

Major parameters :	Lb=220mm	Ct=0mm	P=0mm
	Li=220mm	Cc=0mm	G=100mm
	E1t=55mm	E2t=0mm	
	E1c=55mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	6mm welds on inside and outside of end-plate
Column fastening :	4 × M24 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	294	-

Average pretension force of bolts : 1st row 62 kN
2nd row 58 kN

Failure moment: Max. recorded moment 101 kNm
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B59 : Moment-Rotation Data : Zoetemeijer - Test 32

Moment (kNm)	Rotation (radians)
0.000	0.0000
13.895	0.0003
17.865	0.0010
39.700	0.0026
62.525	0.0055
59.550	0.0093
65.505	0.0131
72.425	0.0194
80.000	0.0270
89.325	0.0376
98.257	0.0496

Table B60 : Moment-Rotation Data : Zoetemeijer - Test 33 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 33

Beam size :	IPE 400
Column size :	HE 450M
End-plate :	400 × 180 × 16 mm

Major parameters :	Lb=290mm	Ct=0mm	P=70mm
	Li=220mm	Cc=0mm	G=100mm
	E1t=55mm	E2t=125mm	
	E1c=55mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	6mm welds on inside and outside of end-plate
Column fastening :	6 × M24 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	286	-

Average pretension force of bolts :	1st row 90 kN
	2nd row 80 kN
	3rd row 86 kN

Failure moment:	Max. recorded moment 246 kNm
Failure mode :	Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B60 : Moment-Rotation Data : Zoetemeijer - Test 33

Moment (kNm)	Rotation (radians)
0.000	0.0000
15.880	0.0002
40.000	0.0004
80.000	0.0010
120.000	0.0024
131.010	0.0028
145.900	0.0040
160.785	0.0055
167.240	0.0063
177.657	0.0081
200.000	0.0131
220.335	0.0197
231.252	0.0250
238.200	0.0310
244.155	0.0375
246.000	0.0432
248.125	0.0458

Table B61 : Moment-Rotation Data : Zoetemeijer - Test 34 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 34

Beam size :	IPE 400
Column size :	HE 450M
End-plate :	400 × 180 × 16 mm

Major parameters :	Lb=290mm	Ct=0mm	P=0mm
	Li=290mm	Cc=0mm	G=100mm
	E1t=55mm	E2t=0mm	
	E1c=55mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	6mm welds on inside and outside of end-plate
Column fastening :	4 × M24 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	286	-

Average pretension force of bolts : 1st row 70 kN
2nd row 68 kN

Failure moment:	Max. recorded moment 208 kNm
Failure mode :	Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B61 : Moment-Rotation Data : Zoetemeijer - Test 34

Moment (kNm)	Rotation (radians)
0.000	0.0000
15.000	0.0002
40.000	0.0008
80.000	0.0021
92.300	0.0025
106.700	0.0034
120.000	0.0048
131.505	0.0060
145.000	0.0085
160.000	0.0125
175.675	0.0167
194.530	0.0251
200.000	0.0284
205.000	0.0325
208.925	0.0377

Table B62 : Moment-Rotation Data : Zoetemeijer - Test 35 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 35

Beam size :	IPE 400
Column size :	HE 450M
End-plate :	400 × 180 × 16 mm

Major parameters :	Lb=220mm	Ct=0mm	P=0mm
	Li=220mm	Cc=0mm	G=100mm
	E1t=125mm	E2t=0mm	
	E1c=55mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	6mm welds on inside and outside of end-plate
Column fastening :	4 × M24 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	286	-

Average pretension force of bolts : 1st row 70 kN
2nd row 74 kN

Failure moment: Max. recorded moment 129 kNm
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B62 : Moment-Rotation Data : Zoetemeijer - Test 35

Moment (kNm)	Rotation (radians)
0.000	0.0000
13.000	0.0004
17.500	0.0012
40.000	0.0026
53.100	0.0044
65.505	0.0065
80.395	0.0109
95.280	0.0163
106.200	0.0266
117.115	0.0364
127.040	0.0500

Table B63 : Moment-Rotation Data : Zoetemeijer - Test 36 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 36

Beam size :	IPE 400
Column size :	HE 450M
End-plate :	400 × 180 × 32 mm

Major parameters :	Lb=345mm	Ct=0mm	P=70mm
	Li=220mm	Cc=0mm	G=100mm
	E1t=55mm	E2t=125mm	
	E1c=55mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	10 mm welds on inside and outside of end-plate
Column fastening :	6 × M24 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	240	-

Average pretension force of bolts : 1st row 20 kN
2nd row 56 kN
3rd row not measured

Failure moment:	Max. recorded moment 306 kNm
Failure mode :	Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B63 : Moment-Rotation Data : Zoetemeijer - Test 36

Moment (kNm)	Rotation (radians)
0.000	0.0000
40.000	0.0006
80.000	0.0016
120.000	0.0021
160.000	0.0031
200.000	0.0050
220.335	0.0063
240.000	0.0084
250.100	0.0097
263.010	0.0120
279.000	0.0164
286.340	0.0258
300.725	0.0258

Table B64 : Moment-Rotation Data : Zoetemeijer - Test 37 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 37

Beam size :	IPE 400
Column size :	HE 450M
End-plate :	400 × 180 × 32 mm

Major parameters :	Lb=290mm	Ct=0mm	P=0mm
	Li=290mm	Cc=0mm	G=100mm
	E1t=55mm	E2t=0mm	
	E1c=55mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	10 mm welds on inside and outside of end-plate
Column fastening :	6 × M24 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	240	-

Average pretension force of bolts : 1st row 34 kN
2nd row 32 kN

Failure moment: Max. recorded moment 213 kNm
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B64 : Moment-Rotation Data : Zoetemeijer - Test 37

Moment (kNm)	Rotation (radians)
0.000	0.0000
25.805	0.0004
40.000	0.0012
50.120	0.0015
80.000	0.0024
120.000	0.0038
160.000	0.0053
200.000	0.0081
231.250	0.0097
198.500	0.0112
187.580	0.0121
141.927	0.0137

Table B65 : Moment-Rotation Data : Zoetemeijer - Test 38 Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P [3.13]
Test identification:	Test 38

Beam size :	IPE 400
Column size :	HE 450M
End-plate :	400 × 180 × 32 mm

Major parameters :	Lb=220mm	Ct=0mm	P=0mm
	Li=220mm	Cc=0mm	G=100mm
	E1t=125mm	E2t=0mm	
	E1c=55mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	10 mm welds on inside and outside of end-plate
Column fastening :	4 × M24 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	240	-

Average pretension force of bolts : 1st row 54 kN
2nd row 36 kN

Failure moment:	Max. recorded moment
Failure mode :	Failure stage not reached

Remarks: 1) Moment-rotation curve supplied privately
2) Very stiff column flange

Table B65 : Moment-Rotation Data : Zoetemeijer - Test 38

Moment (kNm)	Rotation (radians)
0.000	0.0000
24.815	0.0006
49.625	0.0020
64.010	0.0022
101.235	0.0057
120.000	0.0076
138.950	0.0105
150.860	0.0117
163.760	0.0170
168.725	0.0259
166.740	0.0317
147.882	0.0434

Table B66 : Moment-Rotation Data : Phillips and Packer - Test BM1
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Zoetemeijer, P [3.14]
Test identification:	Test BM1

Beam size :	W250 × 33
Column size :	W200 × 100
End-plate :	280 × 160 × 9.5 mm

Major parameters :	Lb=170mm	Ct= -	P=60mm
	Li=110mm	Cc= -	G=80mm
	E1t+Ct=55mm	E2t+Ct=115mm	
	E1c+Cc=55mm	E2c=0mm	

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	Weld
Column fastening :	6 bolts A325 7/8in. diameter(M22)
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	363	-
Column flange:	-	-
Column web:	-	-
End-plate:	300	-

Average pretension force of bolts : 1st bolt row 188kN
2nd bolt row 178kN

Failure moment: 120 kNm
Failure mode : Stopped due to excesive deflection

Remarks: 1) Moment-rotation data derived from M-O curves
2) Column rigidly fixed along the other flange

Table B66 : Moment-Rotation Data : Phillips and Packer - Test BM1

Moment (kNm)	Rotation (radians)
0.00	0.0000
44.50	0.0020
73.50	0.0040
85.10	0.0060
91.00	0.0080
94.80	0.0100
97.70	0.0120
100.60	0.0140
102.20	0.0140
103.00	0.0180
104.50	0.0200
106.40	0.0220
107.60	0.0240
109.40	0.0260
110.30	0.0280
111.30	0.0300
112.30	0.0320
114.20	0.0340
115.20	0.0360
116.10	0.0380
116.20	0.0400

Table B67 : Moment-Rotation Data : Phillips and Packer - Test BM2
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Zoetemeijer, P [3.14]
Test identification:	Test BM2

Beam size :	W250 × 33
Column size :	W200 × 100
End-plate :	280 × 160 × 15.9 mm

Major parameters :	Lb=170mm	Ct= -	P=60mm
	Li=110mm	Cc= -	G=80mm
	E1t+Ct=55mm	E2t+Ct=115mm	
	E1c+Cc=55mm	E2c=0mm	

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	Weld
Column fastening :	6 bolts A325 7/8in. diameter(M22)
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	363	-
Column flange:	-	-
Column web:	-	-
End-plate:	285	-

Average pretension force of bolts : 1st bolt row 180kN
2nd bolt row 175kN

Failure moment: 140 kNm
Failure mode : Bolt fracture in top row of tension region

Remarks: 1) Moment-rotation data derived from M-O curves
2) Column rigidly fixed along the other flange

Table B67 : Moment-Rotation Data : Phillips and Packer - Test BM2

Moment (kNm)	Rotation (radians)
0.00	0.0000
66.80	0.0020
92.90	0.0040
101.60	0.0060
106.50	0.0080
110.30	0.0100
113.70	0.0120
116.10	0.0140
118.50	0.0160
121.00	0.0180
122.90	0.0200
124.50	0.0220
125.80	0.0240
127.80	0.0260
130.00	0.0280
131.20	0.0300
132.60	0.0320
133.60	0.0340
135.50	0.0360
136.50	0.0380
137.50	0.0400

Table B68 : Moment-Rotation Data : Phillips and Packer - Test BM3
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Zoetemeijer, P [3.14]
Test identification:	Test BM3

Beam size :	W250 × 33
Column size :	W200 × 100
End-plate :	280 × 160 × 19.1mm

Major parameters :	Lb=170mm	Ct= -	P=60mm
	Li=110mm	Cc= -	G=80mm
	E1t+Ct=55mm	E2t+Ct=115mm	
	E1c+Cc=55mm	E2c=0mm	

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	Weld
Column fastening :	6 bolts A325 7/8in. diameter(M22)
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	363	-
Column flange:	-	-
Column web:	-	-
End-plate:	259	-

Average pretension force of bolts : 1st bolt row 185kN
2nd bolt row 185kN

Failure moment: 140 kNm
Failure mode : Bolt fracture in top row of tension region

Remarks: 1) Moment-rotation data derived from M-O curves
2) Column rigidly fixed along the other flange

Table B68 : Moment-Rotation Data : Phillips and Packer - Test BM3

Moment (kNm)	Rotation (radians)
0.00	0.0000
77.40	0.0020
98.10	0.0040
106.50	0.0060
112.30	0.0080
116.10	0.0100
119.50	0.0120
121.90	0.0140
123.90	0.0160
125.80	0.0180
128.20	0.0200
130.20	0.0220
132.10	0.0240
133.60	0.0260
135.50	0.0280
137.40	0.0300
138.40	0.0320
139.40	0.0340
140.50	0.0360
141.30	0.0380
142.00	0.0400

Table B69 : Moment-Rotation Data : Phillips and Packer - Test BM4
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Zoetemeijer, P [3.14]
Test identification:	Test BM4

Beam size :	W250 × 33
Column size :	W200 × 100
End-plate :	280 × 160 × 22.2 mm

Major parameters :	Lb=170mm	Ct= -	P=60mm
	Li=110mm	Cc= -	G=80mm
	E1t+Ct=55mm	E2t=115mm	
	E1c+Cc=55mm	E2c=0mm	

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	Weld
Column fastening :	6 bolts A325 7/8in. diameter(M22)
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	363	-
Column flange:	-	-
Column web:	-	-
End-plate:	264	-

Average pretension force of bolts : 1st bolt row 184kN
2nd bolt row 180kN

Failure moment: 146 kNm
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation data derived from M-O curves
2) Column rigidly fixed along the other flange

Table B69 : Moment-Rotation Data : Phillips and Packer - Test BM4

Moment (kNm)	Rotation (radians)
0.00	0.0000
87.10	0.0020
104.50	0.0040
112.30	0.0060
118.10	0.0080
123.90	0.0100
125.80	0.0120
128.70	0.0140
131.30	0.0160
133.50	0.0180
135.50	0.0200
137.40	0.0220
139.40	0.0240
141.10	0.0260
142.30	0.0280
143.20	0.0300
145.20	0.0320
145.50	0.0340

Table B70 : Moment-Rotation Data : Phillips and Packer - Test BM5
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Zoetemeijer, P [3.14]
Test identification:	Test BM5

Beam size :	W250 × 33
Column size :	W200 × 100
End-plate :	280 × 160 × 25.4 mm

Major parameters :	Lb=170mm	Ct= -	P=60mm
	Li=110mm	Cc= -	G=80mm
	E1t+Ct=55mm	E2t+Ct=115mm	
	E1c+Cc=55mm	E2c=0mm	

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	Weld
Column fastening :	6 bolts A325 7/8in. diameter(M22)
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	363	-
Column flange:	-	-
Column web:	-	-
End-plate:	264	-

Average pretension force of bolts : 1st bolt row 180kN
2nd bolt row 180kN

Failure moment: 154 kNm
Failure mode : Bolt fracture in top row of tension region

Remarks: 1) Moment-rotation data derived from M-Θ curves
2) Column rigidly fixed along the other flange

Table B70 : Moment-Rotation Data : Phillips and Packer - Test BM5

Moment (kNm)	Rotation (radians)
0.00	0.0000
92.90	0.0020
114.20	0.0040
122.90	0.0060
127.70	0.0080
131.60	0.0100
135.50	0.0120
137.70	0.0140
140.90	0.0160
144.20	0.0180
147.10	0.0200
149.00	0.0220
151.00	0.0240

Table B71 : Moment-Rotation Data : Bose - Test C
Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Bose , D [3.15]
Test identification:	Test C

Beam size :	457 × 191 × 67 UB
Column size :	305 × 305 × 97 UC
End-plate :	400 × 200 × 25 mm

Major parameters :	Lb=300mm	Ct= -	P=75mm
	Li=75mm	Cc= -	G=140mm
	E1t=50mm	E2t=125mm	
	E1c=50mm	E2c=125mm	

Stiffeners :	No
Beam fastening :	weld
Column fastening :	10 × M20 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : Tightened by podger spanner

Failure moment: -
Failure mode : -

Remarks: 1) Mean values derived from moment-rotation curves

Table B71 : Moment-Rotation Data : Bose - Test C

Moment (kNm)	Rotation (radians)
0.00	0.00000
10.65	0.00009
20.00	0.00021
40.00	0.00040
60.75	0.00068
80.55	0.00100
92.40	0.00135
100.00	0.00175
106.15	0.00200
114.10	0.00265
120.00	0.00315
126.00	0.00365
129.30	0.00400
131.50	0.00450
133.20	0.00480
137.00	0.00500
138.50	0.00520

Table B72 : Moment-Rotation Data : Morris and Newsome - Test 1

Connection Type : Flush end-plate connection to stiffened column

Tested by :	Morris, L.J. and Newsome, C.P. [3.6]
Test identification:	Test 1

Beam size :	457 × 191 × 67 UB
Column size :	305 × 165 × 40 UC
End-plate :	25 mm

Major parameters :	Lb= -	Ct= -	P= -
	Li= -	Cc= -	G= -
	E1t= -	E2t= -	
	E1c= -	E2c= -	

Stiffeners :	Compression stiffeners on clomn web
Beam fastening :	weld
Column fastening :	8 × 7/8in bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : Tightened by podger spanner

Failure moment:	Maximum recorded moment 153kNm
Failure mode :	Buckling of the web in the column panel

Remarks: 1) M-O data derived from the moment-rotation curve
2) End-plate portudes above and below flange

Table B72 : Moment-Rotation Data : Morris and Newsome - Test 1

Moment (kNm)	Rotation (radians)
0.00	0.00000
34.32	0.00260
51.48	0.00340
68.64	0.00660
85.80	0.00950
102.96	0.01430
112.00	0.01950
120.12	0.02770
127.50	0.03720
137.28	0.04680
144.65	0.05830
149.55	0.06730
152.80	0.08000
153.60	0.09740
152.80	0.10600

Table B73 : Moment-Rotation Data : Morris and Newsome - Test 2

Connection Type : Flush end-plate connection to stiffened column

Tested by : Morris, L.J. and Newsome, C.P. [3.6]
Test identification: Test 2

Beam size : 457 × 191 × 67 UB
Column size : 305 × 165 × 40 UC
End-plate : 25 mm

Major parameters :	Lb= -	Ct= -	P= -
	Li= -	Cc= -	G= -
	E1t= -	E2t= -	
	E1c= -	E2c= -	

Stiffeners : Compression stiffeners on column web and small 12mm glut stiffeners on column w
Beam fastening : weld
Column fastening : 8 × 7/8in bolts
Hole type : -

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 1st row 120 kN
2nd row 95 kN

Failure moment: Maximum recorded moment 167kNm
Failure mode : Buckling of the web in the column

Remarks: 1) M-O data derived from the moment-rotation curve
2) End-plate portudes above and below beam flanges

Table B73 : Moment-Rotation Data : Morris and Newsome - Test 2

Moment (kNm)	Rotation (radians)
0.00	0.00000
34.32	0.00260
51.48	0.00340
68.64	0.00660
85.80	0.00950
102.96	0.01430
112.00	0.01900
120.12	0.02200
127.50	0.03100
137.28	0.04000
144.65	0.05300
152.80	0.06450
161.80	0.07920
165.00	0.08800
166.70	0.10000
166.70	0.11800

Table B74 : Moment-Rotation Data : Morris and Newsome - Test 3
Connection Type : Flush end-plate connection to stiffened column

Tested by : Morris, L.J. and Newsome, C.P. [3.6]
Test identification: Test 3

Beam size : $457 \times 191 \times 67$ UB
Column size : $305 \times 165 \times 40$ UC
End-plate : 25 mm

Major parameters :	Lb= -	Ct= -	P= -
	Li= -	Cc= -	G= -
	E1t= -	E2t= -	
	E1c= -	E2c= -	

Stiffeners : Compression stiffeners on column web and small 12mm K-stiffeners on column we
Beam fastening : weld
Column fastening : $8 \times 7/8$in bolts
Hole type : -

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 1st row 111 kN
2nd row 116 kN

Failure moment: Maximum recorded moment 193kNm
Failure mode : Buckling of the web in the column panel

Remarks: 1) M-O data derived from the moment-rotation curve
2) End-plate portudes above and below beam flanges

Table B74 : Moment-Rotation Data : Morris and Newsome - Test 3

Moment (kNm)	Rotation (radians)
0.00	0.00000
34.32	0.00260
51.48	0.00340
68.64	0.00600
85.80	0.00810
102.96	0.01100
120.12	0.01670
137.28	0.02480
154.44	0.03720
163.85	0.04770
171.60	0.05590
181.40	0.06250
187.15	0.07930
191.20	0.09030
192.85	0.10320

Table B75 : Moment-Rotation Data : Morris and Newsome - Test 4

Connection Type : Flush end-plate connection to stiffened column

Tested by :	Morris, L.J. and Newsome, C.P. [3.6]
Test identification:	Test 4

Beam size :	457 × 191 × 67 UB
Column size :	305 × 165 × 40 UC
End-plate :	25 mm

Major parameters :	Lb= -	Ct= -	P= -
	Li= -	Cc= -	G= -
	E1t= -	E2t= -	
	E1c= -	E2c= -	

Stiffeners :	Compression stiffeners on column web and 10mm 'Morris' stiffeners on column we
Beam fastening :	weld
Column fastening :	8 × 7/8in bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 1st row 130 kN
2nd row 120 kN

Failure moment:	Maximum recorded moment 193kNm
Failure mode :	Buckling of the web in the column panel

Remarks: 1) M-O data derived from the moment-rotation curve
2) End-plate portudes above and below beam flanges

Table B75 : Moment-Rotation Data : Morris and Newsome - Test 4

Moment (kNm)	Rotation (radians)
0.00	0.00000
34.32	0.00260
51.48	0.00340
68.64	0.00600
85.80	0.00810
102.96	0.00980
120.12	0.01340
137.28	0.01860
154.44	0.02770
163.85	0.03920
171.60	0.04770
183.45	0.06300
194.00	0.07880
205.92	0.09670
216.55	0.11460
222.30	0.13228
222.40	0.14660
219.00	0.16000

Table B76 : Moment-Rotation Data : Mann and Morris - Test 1
Connection Type : Flush end-plate connection to stiffened column

Tested by : Mann, A.P and Morris, L.J. [3.14]
Test identification: Test 1

Beam size : 406 × 140 × 46 UB
Column size : 203 × 203 × 71 UC
End-plate : 440 × 200 × 12mm

Major parameters :	Lb=275mm	Ct=32.5mm	P=75mm
	Li=125mm	Cc=32.5mm	G=125mm
	E1t=50mm	E2t=125mm	
	E1c=50mm	E2c=0mm	

Stiffeners : 4 pairs of column web stiffeners opposite both beam flanges
Beam fastening : 8mm fillet weld
Column fastening : 8 M22 HSFG bolts
Hole type : 24mm punched holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 1st bolt row 185kN
2nd bolt row 200kN
2nd bolt row 170kN

Failure moment: -
Failure mode : Weld failure

Remarks: 1) Moment-rotation data was not recorded
2) Perfect fit between end-plate and column flange
3) 3 rows of bolts in tension zone

Table B77 : Moment-Rotation Data : Mann and Morris - Test 2
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Mann, A.P and Morris, L.J. [3.14]
Test identification:	Test 2

Beam size :	406 × 140 × 46 UB
Column size :	203 × 203 × 71 UC
End-plate :	440 × 200 × 12mm

Major parameters :	Lb=275mm	Ct=32.5mm	P=75mm
	Li=125mm	Cc=32.5mm	G=125mm
	E1t=50mm	E2t=125mm	
	E1c=50mm	E2c=0mm	

Stiffeners :	4 pairs of column web stiffeners opposite both beam flanges
Beam fastening :	8mm fillet weld
Column fastening :	8 M22 HSFG bolts
Hole type :	24mm punched holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 1st bolt row 185kN
2nd bolt row 200kN
2nd bolt row 170kN

Failure moment: -
Failure mode : failure stage not reached

Remarks:
1) Moment-rotation data was not recorded
2) Imperfect fit between end-plate and column flange
3) 3 rows of bolts in tension zone

Table B78 : Moment-Rotation Data : Mann and Morris - Test 3
Connection Type : Flush end-plate connection to stiffened column

Tested by : Mann, A.P and Morris, L.J. [3.14]
Test identification: Test 3

Beam size : 406 × 140 × 46 UB
Column size : 203 × 203 × 71 UC
End-plate : 440 × 200 × 12mm

Major parameters :	Lb=275mm	Ct=32.5mm	P=75mm
	Li=125mm	Cc=32.5mm	G=125mm
	E1t=50mm	E2t=125mm	
	E1c=50mm	E2c=0mm	

Stiffeners : 4 pairs of column web stiffeners opposite both beam flanges
Beam fastening : 8mm fillet weld
Column fastening : 8 M22 HSFG bolts
Hole type : 24mm punched holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 1st bolt row 185kN
2nd bolt row 200kN
2nd bolt row 170kN

Failure moment: -
Failure mode : failure stage not reached

Remarks: 1) Moment-rotation data was not recorded
2) Shims intepoosed between the end-plate and column flange
3) 3 rows of bolts in tension zone

Table B79 : Moment-Rotation Data : Mann and Morris - Test 4
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Mann, A.P and Morris, L.J. [3.14]
Test identification:	Test 4

Beam size :	406 × 140 × 46 UB
Column size :	203 × 203 × 71 UC
End-plate :	440 × 200 × 20mm

Major parameters :	Lb=275mm	Ct=32.5mm	P=75mm
	Li=125mm	Cc=32.5mm	G=125mm
	E1t=50mm	E2t=125mm	
	E1c=50mm	E2c=0mm	

Stiffeners :	4 pairs of column web stiffeners opposite both beam flanges
Beam fastening :	8mm fillet welds
Column fastening :	8 M22 HSFG bolts
Hole type :	24mm punched holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts :	1st bolt row 185kN
	2nd bolt row 200kN
	2nd bolt row 170kN

Failure moment:	-
Failure mode :	failure stage not reached

Remarks: 1) Moment-rotation data was not recorded
2) Perfect fit between end-plate and column flange
3) 3 rows of bolts in tension zone

Table B80 : Moment-Rotation Data : Mann and Morris - Test 5
Connection Type : Flush end-plate connection to stiffened column

Tested by : Mann, A.P and Morris, L.J. [3.14]
Test identification: Test 5

Beam size : 406 × 140 × 46 UB
Column size : 203 × 203 × 71 UC
End-plate : 440 × 200 × 20mm

Major parameters :			
	Lb=375mm	Ct=32.5mm	P=75mm
	Li=125mm	Cc=32.5mm	G=125mm
	E1t=50mm	E2t=125mm	
	E1c=50mm	E2c=0mm	

Stiffeners : 4 pairs of column web stiffeners opposite both beam flanges
Beam fastening : 8mm fillet welds
Column fastening : 8 M22 HSFG bolts
Hole type : 24mm punched holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 1st bolt row 185kN
2nd bolt row 200kN
3rd bolt row 170kN

Failure moment: -
Failure mode : End-plate

Remarks: 1) Moment-rotation data was not recorded
2) Imperfect fit between end-plate and column flange (3mm out of flateness)
3) 3 rows of bolts in tension zone

Table B81 : Moment-Rotation Data : Mann and Morris - Test 6
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Mann, A.P and Morris, L.J. [3.14]
Test identification:	Test 6

Beam size :	406 × 140 × 46 UB
Column size :	203 × 203 × 71 UC
End-plate :	440 × 200 × 20mm

Major parameters :	Lb=375mm	Ct=32.5mm	P=75mm
	Li=125mm	Cc=32.5mm	G=125mm
	E1t=50mm	E2t=125mm	
	E1c=50mm	E2c=0mm	

Stiffeners :	4 pairs of column web stiffeners opposite both beam flanges
Beam fastening :	8mm fillet welds
Column fastening :	8 M22 HSFG bolts
Hole type :	24mm punched holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 1st bolt row 185kN
2nd bolt row 200kN
2nd bolt row 170kN

Failure moment: -
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation data was not recorded
2) Shims interposed between end-plate and column flange
3) 3 pairs of bolts in tension zone

Table B82 : Moment-Rotation Data : Tong - Test 1
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Tong, C.S. [3.7]
Test identification:	Test 1

Beam size :	305 × 165 × 40 UB
Column size :	254 × 254 × 89 UC
End-plate :	290 × 200 × 8 mm

Major parameters :	Lb=190mm	Ct=12mm	P=0mm
	Li=190mm	Cc=12mm	G=125mm
	E1t=38mm	E2t=0mm	
	E1c=38mm	E2c=0mm	

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	4 × M20 grade 8.8 bolts
Hole type :	32mmm drilled holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 25 kN

Failure moment:	30.5kNm
Failure mode :	End-plate

Remarks: 1) M-O data derived from moment-rotation curve

Table B82 : Moment-Rotation Data : Tong - Test 1

Moment (kNm)	Rotation (radians)
0.00	0.0000
10.00	0.0010
15.00	0.0020
19.00	0.0030
20.50	0.0040
22.00	0.0050
24.00	0.0060
25.20	0.0070
26.40	0.0080
27.00	0.0090
27.60	0.0100
28.50	0.0110
29.00	0.0120
29.70	0.0130
30.10	0.0140
30.50	0.0150

Table B83 : Moment-Rotation Data : Tong - Test 3
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Tong, C.S. [3.7]
Test identification:	Test 3

Beam size :	305 × 165 × 40 UB
Column size :	254 × 254 × 132 UC
End-plate :	340 × 240 × 12 mm

Major parameters :	Lb=175mm	Ct=15mm	P=70mm
	Li=105mm	Cc=15mm	G=120mm
	E1t=65mm	E2t=135mm	
	E1c=65mm	E2c=0mm	

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 grade 8.8 bolts
Hole type :	22mmm drilled holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 25 kN

Failure moment: 128.0kNm
Failure mode : Weld at tension flange

Remarks: 1) M-O data derived from moment-rotation curve
2) 2 rows of bolts in tension region

Table B83 : Moment-Rotation Data : Tong - Test 3

Moment (kNm)	Rotation (radians)
0.00	0.000
15.70	0.001
32.20	0.002
43.50	0.003
59.80	0.005
71.30	0.007
75.90	0.008
81.00	0.010
85.00	0.012
87.40	0.014
89.00	0.016
90.00	0.018
91.00	0.020
92.00	0.022
93.00	0.025

Table B84 : Moment-Rotation Data : Tong - Test 5
Connection Type : Flush end-plate connection to stiffened column

Tested by : Tong, C.S. [3.7]
Test identification: Test 5

Beam size : 305 × 165 × 54 UB
Column size : 254 × 254 × 132 UC
End-plate : 340 × 240 × 25 mm

Major parameters :	Lb=175mm	Ct=15mm	P=70mm
	Li=105mm	Cc=15mm	G=120mm
	E1t=65mm	E2t=135mm	
	E1c=65mm	E2c=0mm	

Stiffeners : Column web stiffeners opposite both beam flanges
Beam fastening : 10mm fillet weld
Column fastening : 6 × M20 grade 8.8 bolts
Hole type : 22mmm drilled holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 25 kN

Failure moment: 165.0kNm
Failure mode : 1st bolt row

Remarks: 1) M-O data derived from moment-rotation curve
2) 2 rows of bolts in tension zone

Table B84 : Moment-Rotation Data : Tong - Test 5

Moment (kNm)	Rotation (radians)
0.00	0.0000
20.10	0.0010
40.20	0.0020
59.80	0.0030
80.00	0.0040
100.00	0.0050
119.50	0.0060
133.30	0.0070
140.00	0.0080
147.70	0.0100
151.70	0.0120
154.50	0.0140
156.30	0.0160
158.60	0.0190
160.90	0.0210
162.00	0.0230
163.20	0.0250
164.90	0.0252

Table B85 : Moment-Rotation Data : Tong - Test 12
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Tong, C.S. [3.7]
Test identification:	Test 12

Beam size :	305 × 165 × 54 UB
Column size :	254 × 254 × 132 UC
End-plate :	340 × 200 × 12 mm

Major parameters :	Lb=175mm	Ct=15mm	P=60mm
	Li=115mm	Cc=15mm	G=100mm
	E1t=65mm	E2t=125mm	
	E1c=65mm	E2c=0mm	

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 grade 8.8 bolts
Hole type :	22mm drilled holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	300	-

Average pretension force of bolts : 25 kN

Failure moment:	Max. recorded moment 111.8kNm
Failure mode :	Failure stage not reached

Remarks: 1) M-O data derived from moment-rotation curve
2) 2 rows of bolts in tension zone

Table B85 : Moment-Rotation Data : Tong - Test 12

Moment (kNm)	Rotation (radians)
0.00	0.00000
10.60	0.00056
20.40	0.00112
39.90	0.00248
50.30	0.00336
60.20	0.00496
70.80	0.00680
79.75	0.00925
84.65	0.01187
91.20	0.01696
93.60	0.02352

Table B86 : Moment-Rotation Data : Tong - Test 16
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Tong, C.S. [3.7]
Test identification:	Test 16

Beam size :	305 × 165 × 54 UB
Column size :	254 × 254 × 132 UC
End-plate :	240 × 340 × 15 mm

Major parameters :	Lb=175mm	Ct=15mm	P=70mm
	Li=105mm	Cc=15mm	G=120mm
	E1t=65mm	E2t=135mm	
	E1c=65mm	E2c=0mm	

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 grade 8.8 bolts
Hole type :	22mm drilled holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 25 kN

Failure moment: 160.0kNm
Failure mode : 1st bolt row

Remarks: 1) M-O data derived from moment-rotation curve
2) 2 rows of bolts in tension zone

Table B86 : Moment-Rotation Data : Tong - Test 16

Moment (kNm)	Rotation (radians)
0.00	0.00000
21.20	0.00088
32.70	0.00144
40.90	0.00220
53.20	0.00288
61.40	0.00355
72.00	0.00440
84.30	0.00584
91.60	0.00668
102.30	0.00855
106.40	0.00981
111.30	0.01142
117.80	0.01345
121.10	0.01633
127.60	0.02115
129.30	0.02555

Table B87 : Moment-Rotation Data : Tong - Test 17
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Tong, C.S. [3.7]
Test identification:	Test 17

Beam size :	305 × 165 × 54 UB
Column size :	254 × 254 × 132 UC
End-plate :	340 × 240 × 20 mm

Major parameters :	Lb=175mm	Ct=15mm	P=70mm
	Li=105mm	Cc=15mm	G=120mm
	E1t=65mm	E2t=135mm	
	E1c=65mm	E2c=0mm	

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 grade 8.8 bolts
Hole type :	22mm drilled holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 25 kN

Failure moment: 168.0kNm
Failure mode : 1st bolt row

Remarks: 1) M-O data derived from moment-rotation curve
2) 2 pairs of bolts in tension zone

Table B87 : Moment-Rotation Data : Tong - Test 17

Moment (kNm)	Rotation (radians)
0.00	0.00000
32.70	0.00110
42.60	0.00152
53.20	0.00203
62.20	0.00253
72.00	0.00304
83.40	0.00372
95.70	0.00457
116.20	0.00694
137.50	0.01049
144.00	0.01286
150.50	0.01540
155.50	0.02048
158.70	0.02538

Table B88 : Moment-Rotation Data : Zandonini and Zanon - Test FP1-1
Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zandonini, R. and Zanon, P. [3.3]
Test identification:	Test FP1-1

Beam size :	IPE300
Column size :	Rigid counterbeam
End-plate :	280 × 170 × 12 mm

Major parameters :	Lb=120mm	Ct=0mm	P=0mm
	Li=120mm	Cc=0mm	G=105mm
	E1t=80mm	E2t=0mm	
	E1c=80mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	4 × M20 grade 4.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	304.5	436.0
Beam web:	327.0	458.o
Column flange:	-	-
Column web:	-	-
End-plate:	356.0	575.5

Average pretension force of bolts : Preload to 40

Failure moment: 65.70kNm
Failure mode : Failure of bolt in tension

Remarks: 1) M-O data derived from moment-rotation curve

Table B88 : Moment-Rotation Data : Zandonini and Zanon - Test FP1-1

Moment (kNm)	Rotation (radians)
0.00	0.00000
20.00	0.00138
28.40	0.00276
34.00	0.00413
40.00	0.00552
45.00	0.00752
50.00	0.00965
54.20	0.01207
57.00	0.01517
60.00	0.02000
64.00	0.02655
66.00	0.03310
67.00	0.03603
67.60	0.04000
67.20	0.04552
65.60	0.05138
63.60	0.05621
62.60	0.06000
60.60	0.06250
57.20	0.06690

Table B89 : Moment-Rotation Data : Zandonini and Zanon - Test FP1-2
Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zandonini, R. and Zanon, P. [3.3]
Test identification:	Test FP1-2

Beam size :	IPE300
Column size :	Rigid counterbeam
End-plate :	280 × 170 × 12 mm

Major parameters :	Lb=120mm	Ct=0mm	P=60mm
	Li=60mm	Cc=0mm	G=105mm
	E1t=80mm	E2t=0mm	
	E1c=80mm	E2c=0mm	

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	6 × M20 grade 4.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	304.5	436.0
Beam web:	327.0	458.0
Column flange:	-	-
Column web:	-	-
End-plate:	356.0	575.5

Average pretension force of bolts : Preload to 40

Failure moment: 80.90kNm
Failure mode : Bolt fracture

Remarks: 1) M-O data derived from moment-rotation curve

Table B89 : Moment-Rotation Data : Zandonini and Zanon - Test FP1-2

Moment (kNm)	Rotation (radians)
0.00	0.00000
20.00	0.00170
30.00	0.00340
40.00	0.00511
50.00	0.00750
60.00	0.01160
66.80	0.01570
70.00	0.02000
74.40	0.02591
77.20	0.03136
79.60	0.04000
81.00	0.04398
82.00	0.05148
82.40	0.06000
82.00	0.07023
81.20	0.08000
80.00	0.08523

Table B90 : Moment-Rotation Data : Zandonini and Zanon - Test FP1-3
Connection Type : Flush end-plate connection to unstiffened column

Tested by :	Zandonini, R. and Zanon, P. [3.3]
Test identification:	Test FP1-3

Beam size :	IPE300
Column size :	Rigid counterbeam
End-plate :	280 × 170 × 12 mm

Major parameters :	Lb=180mm	Ct=0mm	P=60mm
	Li=60mm	Cc=0mm	G=105mm
	E1t=50mm	E2t=110mm	
	E1c=50mm	E2c=110mm	

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	8 × M20 grade 4.8 bolts
Hole type :	22mmm drilled holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	304.5	436.0
Beam web:	327.0	458.0
Column flange:	-	-
Column web:	-	-
End-plate:	356.0	575.5

Average pretension force of bolts : Preload to 40

Failure moment: 81.10kNm
Failure mode : Bolt stripping

Remarks: 1) M-O data derived from moment-rotation curve

Table B90 : Moment-Rotation Data : Zandonini and Zanon - Test FP1-3

Moment (kNm)	Rotation (radians)
0.00	0.00000
10.00	0.00308
20.00	0.00583
30.00	0.00686
40.00	0.00892
50.00	0.01098
60.00	0.01303
70.00	0.01578
75.40	0.01818
80.00	0.02000
84.00	0.02367
86.50	0.02641
88.30	0.03156
89.00	0.03512
88.50	0.04000
84.60	0.04390
80.00	0.04596
74.00	0.04733

Table B91 : Moment-Rotation Data : Davison - Test JT/12 Connection Type : Flush end-plate connection to unstiffened column

Tested by : Davison, J.B. [3.20]
Test identification: Test JT/12

Beam size : 254 × 102 × 22 UB
Column size : 152 × 152 × 23 UC
End-plate : 265 × 125 × 12 mm

Major parameters :	Lb=150mm	Ct=5mm	P=50mm
	Li=100mm	Cc=5mm	G=76mm
	E1t=35mm	E2t=85mm	
	E1c=35mm	E2c=0mm	

Stiffeners : No
Beam fastening : 4mm fillet weld
Column fastening : 6 × M16 grade 4.6 bolts
Hole type : 18mm holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	307.0	-
Beam web:	310.6	-
Column flange:	270.9	-
Column web:	266.3	-
End-plate:	290.4	-

Average pretension force of bolts : Torque of approximatly 160Nm

Failure moment: Max. moment recorded 28.47kNm at RHD and 24.74kNm at LHD
Failure mode : Failure stage not reached

Remarks: 1) Moment-rotation provided in numerical form
2) Mean values of RHD and LHD are tabulated

Table B91 : Moment-Rotation Data : Davison - Test JT/12

Moment (kNm)	Rotation (radians)
0.000	0.00000000
2.288	0.00005256
5.434	0.00030360
8.151	0.00050025
9.295	0.00062588
11.154	0.00090427
12.870	0.00109727
13.871	0.00153169
14.729	0.00203589
15.444	0.00217185
16.016	0.00261776
16.731	0.00327243
17.589	0.00396620
19.591	0.00641673
18.590	0.00517487
19.162	0.00610064
19.591	0.00641673
19.877	0.00681367
20.449	0.00829920
21.021	0.00937864
21.593	0.01075804
22.022	0.01167427
22.451	0.01270341
22.737	0.01339613
23.023	0.01400657
23.166	0.01441302
23.452	0.01507607
24.167	0.01644489
24.453	0.01731273
24.739	0.01796151
25.769	0.01353841
26.718	0.01514970
27.156	0.01606287
28.032	0.01739341
28.470	0.01841740

Table B92 : Moment-Rotation Data : Prescott - Test 19
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Prescott, A.T. [3.10]
Test identification:	Test 19

Beam size :	254 × 146 × 43 UB
Column size :	203 × 203 × 71 UC
End-plate :	290 × 170 × 12 mm

Major parameters :	Lb=125mm	Ct=15mm	P=60mm
	Li=65mm	Cc=15mm	G=100mm
	E1t=80mm	E2t=140mm	
	E1c=80mm	E2c=0mm	

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 grade 8.8 bolts
Hole type :	22mm drilled holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	300	-

Average pretension force of bolts : 25 kN

Failure moment: 97.4kNm
Failure mode : Botl fracture

Remarks: 1) M-O data derived from moment-rotation curve

Table B92 : Moment-Rotation Data : Prescott - Test 19

Moment (kNm)	Rotation (radians)
0.00	0.00000
19.00	0.00020
27.00	0.00080
35.00	0.00180
40.00	0.00280
45.00	0.00400
51.00	0.00600
55.50	0.00800
58.50	0.01000
61.50	0.01200
63.00	0.01400
65.50	0.01600
67.00	0.01800
68.50	0.02000
70.00	0.02200
71.50	0.02400
72.50	0.02600
73.50	0.02800
74.00	0.03000
75.00	0.03030

Table B93 : Moment-Rotation Data : Prescott - Test 20
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Prescott, A.T. [3.10]
Test identification:	Test 20

Beam size :	254 × 146 × 43 UB
Column size :	203 × 203 × 71 UC
End-plate :	290 × 170 × 15 mm

Major parameters :	Lb=125mm	Ct=15mm	P=60mm
	Li=65mm	Cc=15mm	G=100mm
	E1t=80mm	E2t=140mm	
	E1c=80mm	E2c=0mm	

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 grade 8.8 bolts
Hole type :	22mm drilled holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 25 kN

Failure moment: 110.9kNm
Failure mode : Botl fracture

Remarks: 1) M-O data derived from moment-rotation curve

Table B93 : Moment-Rotation Data : Prescott - Test 20

Moment (kNm)	Rotation (radians)
0.00	0.00000
13.00	0.00020
21.00	0.00040
26.50	0.00100
30.40	0.00180
37.00	0.00240
45.00	0.00360
50.00	0.00480
57.50	0.00600
63.50	0.00800
68.00	0.01000
71.00	0.01200
74.00	0.01400
76.00	0.01600
78.00	0.01800
79.50	0.02000
81.00	0.02200
83.00	0.02400
84.50	0.02600
85.50	0.02800
86.00	0.03000
87.00	0.03200
88.00	0.03385

Table B94 : Moment-Rotation Data : Prescott - Test 21
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Prescott, A.T. [3.10]
Test identification:	Test 21

Beam size :	254 × 146 × 43 UB
Column size :	203 × 203 × 71 UC
End-plate :	290 × 170 × 20 mm

Major parameters :	Lb=125mm	Ct=15mm	P=60mm
	Li=65mm	Cc=15mm	G=100mm
	E1t=80mm	E2t=140mm	
	E1c=80mm	E2c=0mm	

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 grade 8.8 bolts
Hole type :	22mm drilled holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 25 kN

Failure moment: 119.7kNm
Failure mode : Botl fracture

Remarks: 1) M-O data derived from moment-rotation curve

Table B94 : Moment-Rotation Data : Prescott - Test 21

Moment (kNm)	Rotation (radians)
0.00	0.00000
11.00	0.00018
18.00	0.00020
26.50	0.00095
40.00	0.00235
50.00	0.00350
58.00	0.00456
63.50	0.00535
69.50	0.00635
74.50	0.00735
80.00	0.00888
85.50	0.01135
88.00	0.01300
90.00	0.01475
96.00	0.01937
98.50	0.02250

Table B95 : Moment-Rotation Data : Prescott - Test 24
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Prescott, A.T. [3.10]
Test identification:	Test 24

Beam size :	254 × 146 × 43 UB
Column size :	203 × 203 × 71 UC
End-plate :	290 × 170 × 12 mm

Major parameters :	Lb=125mm	Ct=15mm	P=60mm
	Li=65mm	Cc=15mm	G=100mm
	E1t=80mm	E2t=140mm	
	E1c=80mm	E2c=0mm	

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 grade 8.8 bolts
Hole type :	22mm drilled holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	300	-

Average pretension force of bolts : 100 kN

Failure moment: 85.6kNm
Failure mode : Botl fracture

Remarks: 1) M-O data provided in numerical form

Table B95 : Moment-Rotation Data : Prescott - Test 24

Moment (kNm)	Rotation (radians)
0.00	0.00000
2.63	0.00011
5.35	0.00028
10.70	0.00058
16.05	0.00090
21.40	0.00123
26.75	0.00156
32.10	0.00203
37.45	0.00245
42.80	0.00315
48.15	0.00424
50.83	0.00517
53.50	0.00622
56.18	0.00764
57.51	0.00847
58.85	0.00929
60.19	0.01038
61.53	0.01140
62.86	0.01236
64.20	0.01392
65.54	0.01539
66.88	0.01684
68.21	0.01850
69.55	0.02141
70.90	0.02432
72.20	0.02685
73.63	0.02951

Table B96 : Moment-Rotation Data : Prescott - Test 25
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Prescott, A.T. [3.10]
Test identification:	Test 25

Beam size :	254 × 146 × 43 UB
Column size :	203 × 203 × 71 UC
End-plate :	290 × 170 × 15 mm

Major parameters :	Lb=125mm	Ct=15mm	P=60mm
	Li=65mm	Cc=15mm	G=100mm
	E1t=80mm	E2t=140mm	
	E1c=80mm	E2c=0mm	

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 grade 8.8 bolts
Hole type :	22mm drilled holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 100 kN

Failure moment: 107.1kNm
Failure mode : Botl fracture

Remarks: 1) M-O data provided in numerical form

Table B96 : Moment-Rotation Data : Prescott - Test 25

Moment (kNm)	Rotation (radians)
0.00	0.00000
2.68	0.00003
5.35	0.00004
8.03	0.00007
10.70	0.00011
13.38	0.00018
16.05	0.00018
18.73	0.00020
21.40	0.00027
24.08	0.00036
26.75	0.00048
29.43	0.00060
32.10	0.00075
34.78	0.00094
37.45	0.00110
40.13	0.00126
42.80	0.00151
45.48	0.00174
48.15	0.00200
50.83	0.00235
53.50	0.00271
56.18	0.00307
58.85	0.00358
61.53	0.00414
64.20	0.00491
66.88	0.00059
69.55	0.00700
72.20	0.00086
74.90	0.01041
77.58	0.01278
80.25	0.01561
82.93	0.02066
85.60	0.02646

Table B97 : Moment-Rotation Data : Prescott - Test 26
Connection Type : Flush end-plate connection to stiffened column

Tested by :	Prescott, A.T. [3.10]
Test identification:	Test 26

Beam size :	254 × 146 × 43 UB
Column size :	203 × 203 × 71 UC
End-plate :	290 × 170 × 20 mm

Major parameters :	Lb=125mm	Ct=15mm	P=60mm
	Li=65mm	Cc=15mm	G=100mm
	E1t=80mm	E2t=140mm	
	E1c=80mm	E2c=0mm	

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 grade 8.8 bolts
Hole type :	22mm drilled holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 100 kN

Failure moment:	117.7kNm
Failure mode :	Botl fracture

Remarks: 1) M-O data provided in numerical form

Table B97 : Moment-Rotation Data : Prescott - Test 26

Moment (kNm)	Rotation (radians)
0.00	0.00000
5.35	0.00005
10.70	0.00007
16.05	0.00011
21.40	0.00027
26.75	0.00039
32.10	0.00058
37.45	0.00078
42.80	0.00105
48.15	0.00144
53.50	0.00195
56.18	0.00213
58.85	0.00254
61.53	0.00283
64.20	0.00331
66.88	0.00379
69.55	0.00440
72.20	0.00522
74.90	0.00610
77.58	0.00740
80.25	0.00869
82.93	0.01035
85.60	0.01231
88.28	0.01379
90.95	0.01601
93.63	0.01842
96.30	0.02219

Table B98 : Moment-Rotation Data : Chakrabarti - Test 1 and 2 Connection Type : Flush end-plate connection to unstiffened column

Tested by : Chakrabarti, B. [3.21]
Test identification: Tests 1 and 2

Beam size : 254 × 146 × 37 UB
Column size : 152 × 152 × 37 UC
End-plate : 290 × 154 × 12 mm

Major parameters :	Lb=125mm	Ct=15mm	P=60mm
	Li=65mm	Cc=15mm	G=100mm
	E1t=65mm	E2t=125mm	
	E1c=60mm	E2c=0mm	

Stiffeners : No
Beam fastening : 8mm fillet weld
Column fastening : 6 × M16 grade 8.8 bolts
Hole type : 18mm drilled holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : Hand tightened nuts and bolts

Failure moment: 66kNm
Failure mode : Excessive yielding of column flanges

Remarks: 1) Average moment-rotation data is derived from mean values of the two M-O curves (Left and Right)

Table B98: Moment-Rotation Data : Chakrabati - Tests 1 and 2

Moment (kNm)	Rotation (radians)
0.00	0.00000
2.00	0.00050
3.95	0.00080
5.70	0.00105
7.70	0.00155
10.00	0.00171
15.50	0.00263
20.00	0.00390
25.20	0.00500
30.00	0.00684
35.10	0.01000
40.00	0.01421
44.60	0.02000
49.80	0.03000
52.30	0.04000
55.00	0.05000
57.50	0.06000
59.30	0.07000
61.10	0.08000
62.40	0.09000
63.30	0.10000
64.15	0.11000
65.00	0.12000
65.50	0.12673

APPENDIX C

EXTENDED END-PLATE CONNECTIONS

Table C1 : Moment-Rotation Data : Johnson - Test 5

Connection Type : Extended end-plate connection to stiffened column

Tested by : Johnson,L.G. ,Cannon,J.C. and Spooner,L.A. [4.22]
Test identification: Test 5

Beam size : 10 × 4.5 UB 25
Column size : 8 × 8 × UC 45
End-plate : 0.5 in.

Major parameters :			
	Ct= -	Pt=3.759in	Pit=-
	Cc= -	Pc=1.875in	Pic=-
	Lb= -	G=3.500in	Pi= -

Stiffeners : 0.5 in. column web stiffeners opposite both beam flanges
Beam fastening : weld
Column fastening : 8 × 3/4 in. diameter H.S. bolts
Hole type : 15/16 in. oversize holes

Measured Material Properties:

	Yield Stress (Ksi)	Ultimate Stress (Ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : End-plate

Remarks: 1) End-plate extended on tension side only
2) Only moment-deflection curve available, moment-rotation data derived from reference [4.23-4.24]

Table C1 : Moment-Rotation Data : Johnson et al - Test 5

Moment (k.ft)	Rotation (Radians)
0.000	0.00000
3.525	0.00017
7.050	0.00033
10.583	0.00050
14.058	0.00067
17.533	0.00083
21.000	0.00100
24.475	0.00117
27.950	0.00133
31.425	0.00150
35.108	0.00170
38.800	0.00190
42.480	0.00210
46.058	0.00233
49.642	0.00257
53.225	0.00280
56.483	0.00313
59.750	0.00347
63.008	0.00380
66.017	0.00410
69.017	0.00440
71.858	0.00485
74.700	0.00530
77.067	0.00575
79.433	0.00620
81.492	0.00675
83.542	0.00730
84.808	0.00780
85.908	0.00840
87.017	0.00900
87.333	0.00945
87.650	0.00990
88.908	0.01045
90.175	0.01100
90.492	0.01165
90.808	0.01230
91.442	0.01277
92.067	0.01323
92.700	0.01370

Table C2 : Moment-Rotation Data : Sherbourne - Test A1 Connection Type : Extended end-plate connection to unstiffened column

Tested by : Sherbourne, A.N. [4.10]
Test identification: Test A1

Beam size : 15×5 UB 42
Column size : $8 \times 8 \times$ UC 35
End-plate : $18.5\times 7 \times 1.25$ in.

Major parameters :	Ct=1.500in	Pt=3.500in	Pit=2.500in
	Cc=0.750in	Pc=1.750in	Pic=2.500in
	Lb=14.500in	G=4.000in	Pi=3.000in

Stiffeners : No
Beam fastening : weld
Column fastening : $8 \times 3/4$ in. diameter black bolts and
$4 \times 3/4$ in. diameter black bolts.
Hole type : 15/16 in. oversize holes

Measured Material Properties:

	Yield Stress (Ton/in^2)	Ultimate Stress (Ton/in^2)
Beam flange:	15.55	29.6
Beam web:	15.80	30.50
Column flange:	15.80	28.45
Column web:	15.90	28.80
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: Collapse load 21.2 ton.
Failure mode : Column yielding and buckling

Remarks:
1) End-plate extended on tension side only
2) Hardened steel washers used under both nut and head
3) $4\times 3/4$ in. diameter black bolts in compression zone
4) Only load-deflection curve available, moment-rotation data derived from reference [4.23,4.24]

Table C2 : Moment-Rotation Data : Sherbourne - Test A1

Moment (k.ft)	Rotation (Radians)
0.000	0.00000
11.667	0.00015
23.333	0.00030
35.000	0.00040
46.667	0.00050
58.333	0.00065
70.000	0.00080
80.208	0.00100
90.417	0.00120
102.083	0.00130
113.750	0.00130
113.750	0.00140
125.417	0.00165
137.083	0.00190
148.750	0.00210
160.417	0.00230
170.625	0.00275
180.833	0.00320
188.125	0.00365
195.417	0.00410
199.792	0.00450
204.167	0.00490
207.083	0.00530
210.000	0.00570
212.917	0.00615
215.833	0.00660
224.583	0.00710
226.042	0.00765
227.500	0.00820
230.417	0.00885
233.333	0.00950
235.275	0.01007
237.225	0.01063
239.167	0.01120
239.900	0.01177
240.625	0.01235
241.350	0.01293
242.083	0.01350

Table C3 : Moment-Rotation Data : Sherbourne - Test A2 Connection Type : Extended end-plate connection to stiffened column

Tested by :	Sherbourne, A.N. [4.10]
Test identification:	Test A2

Beam size :	15 × 5 UB 42
Column size :	8 × 8 × UC 35
End-plate :	18.5× 7 × 1.25 in.

Major parameters :	Ct=1.500in	Pt=3.500in	Pit=2.500in
	Cc=0.750in	Pc=1.750in	Pic=2.500in
	Lb=14.500in	G=4.000in	Pi=3.000in

Stiffeners :	5/16 in. column web stiffeners opposite both beam flanges
Beam fastening :	weld
Column fastening :	8 × 3/4 in. diameter H.T. bolts and 4 × 3/4 in. diameter black bolts
Hole type :	15/16 in. oversize holes

Measured Material Properties:

	Yield Stress (Ton/in^2)	Ultimate Stress (Ton/in^2)
Beam flange:	16.00	31.00
Beam web:	-	30.90
Column flange:	-	29.80
Column web:	17.80	28.30
End-plate:	16.70	29.50

Average pretension force of bolts : -

Failure moment: Collapse load 38.0 ton.
Failure mode : Bolt fracture

Remarks:
1) End-plate extended on tension side only
2) Hardened steel washers used under both nut and head
3) 4× 3/4 in. diameter black bolts in compression zone
4) Only load-deflection curve available,
moment-rotation data derived from reference [4.24,4.24]

Table C3 : Moment-Rotation Data : Sherbourne - Test A2

Moment (k.ft)	Rotation (Radians)
0.000	0.00000
16.858	0.00005
33.717	0.00010
50.575	0.00015
67.433	0.00020
79.167	0.00020
90.892	0.00020
105.550	0.00037
120.208	0.00053
134.867	0.00070
148.067	0.00080
161.258	0.00090
172.983	0.00110
184.708	0.00130
205.233	0.00130
216.967	0.00140
228.692	0.00150
249.217	0.00220
260.943	0.00240
272.667	0.00260
284.400	0.00295
296.125	0.00330
310.783	0.00380
322.517	0.00450
337.175	0.00530
348.900	0.00630
360.625	0.00700
366.492	0.00810
378.217	0.00920
384.083	0.00995
389.950	0.01070
395.808	0.01170
401.675	0.01270
404.608	0.01355
407.542	0.01440
410.475	0.01505
413.400	0.01570
416.333	0.01640
419.267	0.01710
422.200	0.01785
425.133	0.01860
428.067	0.01945
430.992	0.02030
430.992	0.02010
433.992	0.02095

Table C4 : Moment-Rotation Data : Sherbourne - Test A3
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Sherbourne, A.N. [4.10]
Test identification:	Test A3

Beam size :	15 × 5 UB 42
Column size :	8 × 8 × UC 35
End-plate :	18.5× 7 × 0.75 in.

Major parameters :	Ct=1.500in	Pt=3.500in	Pit=2.500in
	Cc=0.750in	Pc=1.750in	Pic=2.500in
	Lb=14.500in	G=4.000in	Pi=3.000in

Stiffeners :	5/8 in. column web stiffeners opposite both beam flanges
Beam fastening :	weld
Column fastening :	8 × 3/4 in. diameter H.T. bolts and 4 × 3/4 in. diameter black bolts
Hole type :	15/16 in. oversize holes

Measured Material Properties:
All sections mild steel to B.S.15

	Yield Stress (Ton/in^2)	Ultimate Stress (Ton/in^2)
Beam flange:	16.50	29.40
Beam web:	16.40	30.90
Column flange:	16.65	29.40
Column web:	18.20	28.80
End-plate:	16.70	29.00

Average pretension force of bolts : -

Failure moment: Collapse load 42.8 ton.
Failure mode : Bolt fracture

Remarks:
1) End-plate extended on tension side only
2) Hardened steel washers used under both nut and head
3) 4× 3/4 in. diameter black bolts in compression zone
4) Only load-deflection curve available, moment-rotation data derived from reference [4.23,4.24]

Table C4 : Moment-Rotation Data : Sherbourne - Test A3

Moment (k.ft)	Rotation (Radians)
0.000	0.00000
23.700	0.00020
44.442	0.00020
65.183	0.00050
88.892	0.00070
112.592	0.00070
130.367	0.00070
143.700	0.00080
157.033	0.00090
180.742	0.00070
201.483	0.00090
222.225	0.00110
245.925	0.00180
266.667	0.00200
280.000	0.00220
293.333	0.00240
317.033	0.00330
340.742	0.00480
349.633	0.00530
358.517	0.00660
373.333	0.00730
385.183	0.00850
389.633	0.00945
394.075	0.01040
405.925	0.01150
414.817	0.01300
426.667	0.01460
432.592	0.01565
438.517	0.01670
444.444	0.01790
448.892	0.01885
453.333	0.01980
457.775	0.02095
462.225	0.02210
465.183	0.02350
468.150	0.02490
471.108	0.02585
474.075	0.02680
477.033	0.02775
480.000	0.02870
481.483	0.02955
482.967	0.03040
487.408	0.03155
491.850	0.03270

Table C5 : Moment-Rotation Data : Sherbourne - Test B1
Connection Type : Extended end-plate connection to stiffened column

Tested by : Sherbourne, A.N. [4.10]
Test identification: Test B1

Beam size : 15 × 5 UB 42
Column size : 8 × 8 × UC 35
End-plate : 19× 7 × 1.00 in.

Major parameters :	Ct=1.500in	Pt=5.000in	Pit=3.000in
	Cc=1.000in	Pc=2.500in	Pic=3.000in
	Lb=14.000in	G=4.000in	Pi=3.000in

Stiffeners : 5/16 in. column web stiffeners opposite both beam flanges
Beam fastening : weld
Column fastening : 10 × 7/8 in. diameter H.T. bolts
Hole type : -

Measured Material Properties:

	Yield Stress (Ton/in^2)	Ultimate Stress (Ton/in^2)
Beam flange:	16.00	-
Beam web:	-	-
Column flange:	17.63	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: Collapse load 38.2 ton.
Failure mode : Lateral instability of beams and local bucling of column stiffeners

Remarks:
1) End-plate extended on tension side only
2) Hardened steel washers used under both nut and head
3) Only load-deflection curve available, moment-rotation data derived from reference [4.23,4.24]

Table C5 : Moment-Rotation Data : Sherbourne - Test B1

Moment (k.ft)	Rotation (Radians)
0.000	0.00000
21.992	0.00010
43.975	0.00020
63.033	0.00035
82.092	0.00050
96.750	0.00060
111.417	0.00070
124.608	0.00080
137.800	0.00090
152.458	0.00095
167.117	0.00100
180.317	0.00110
193.508	0.00120
205.233	0.00130
216.967	0.00140
237.483	0.00190
249.217	0.00220
269.742	0.00280
323.133	0.00360
293.192	0.00440
305.025	0.00545
316.850	0.00650
325.767	0.00770
337.933	0.00930
346.350	0.01070
354.767	0.01200
360.525	0.01340
366.283	0.01480
378.217	0.01770
384.083	0.01940
389.950	0.02110
398.742	0.02360
404.608	0.02550
407.317	0.02740
411.375	0.02920
413.850	0.03080
416.333	0.03240
425.133	0.03530
425.133	0.03730
428.067	0.03890
430.992	0.04050
432.458	0.04210
433.925	0.04370
439.792	0.04650
439.792	0.04940

Table C6 : Moment-Rotation Data : Sherbourne - Test B2
Connection Type : Extended end-plate connection to stiffened column

Tested by : Sherbourne, A.N. [4.10]
Test identification: Test B2

Beam size : 15 × 5 UB 42
Column size : 8 × 8 × UC 35
End-plate : 19× 7 × 0.75 in.

Major parameters :	Ct=1.500in	Pt=5.000in	Pit=3.000in
	Cc=1.000in	Pc=2.500in	Pic=3.000in
	Lb=14.000in	G=4.000in	Pi=3.000in

Stiffeners : 1/2 in. column web stiffeners opposite both beam flanges
Beam fastening : weld
Column fastening : 10 × 7/8 in. diameter H.T. bolts
Hole type : -

Measured Material Properties:

	Yield Stress (Ton/in^2)	Ultimate Stress (Ton/in^2)
Beam flange:	16.00	-
Beam web:	-	-
Column flange:	17.63	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: Collapse load 37.0 ton.
Failure mode : Weld failure at beam tension and end-plate

Remarks: 1) End-plate extended on tension side only
2) Hardened steel washers used under both nut and head
3) Only load-deflection curve available,
moment-rotation data derived from reference [4.23,4.24]

Table C6 : Moment-Rotation Data : Sherbourne - Test B2

Moment (k.ft)	Rotation (Radians)
0.000	0.00000
16.617	0.00003
33.225	0.00007
49.842	0.00010
65.483	0.00017
81.117	0.00023
96.750	0.00030
111.417	0.00060
129.742	0.00058
148.067	0.00055
166.392	0.00053
184.708	0.00050
196.442	0.00060
208.167	0.00070
228.692	0.00100
249.217	0.00140
269.742	0.00210
282.933	0.00275
296.125	0.00340
313.717	0.00430
328.375	0.00520
343.033	0.00640
351.833	0.00760
363.558	0.00900
369.425	0.00995
375.292	0.01090
384.083	0.01260
392.883	0.01420
395.808	0.01570
400.208	0.01685
404.608	0.01800
407.542	0.01940
410.475	0.02080
413.400	0.02175
416.333	0.02270
419.267	0.02440
419.267	0.02540
419.267	0.02640
422.200	0.02810
425.133	0.02980
428.067	0.03130
430.992	0.03300
430.992	0.03425
430.992	0.03550

Table C7 : Moment-Rotation Data : Mann - Test C1
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Mann, A.P. [4.25]
Test identification:	Test C1

Beam size :	12 × 5 UB 25
Column size :	8 × 8 × UC 48
End-plate :	16.00× 7.00 × 0.75 in.

Major parameters :	Ct=1.500in	Pt=4.500in	Pit=0.000in
	Cc+Pc=3.000in	Pc= -	Pic=0.000in
	Lb=11.500in	G=3.500in	Pi=7.000in

Stiffeners :	No
Beam fastening :	1/16 in. butt welds
Column fastening :	6 × 3/4 in. diameter HSFG bolts
Hole type :	15/16 in. oversize holes

Measured Material Properties:

	Yield Stress (Ton/in^2)	Ultimate Stress (Ton/in^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : Tension bolts preloaded

Failure moment: 98.545 kips.ft
Failure mode : Weld fracture

Remarks: 1) End-plate extended on tension side only
2) 7 in. × 3 in. shear plate welded to column to support end-plate
4) Moment -rotation data derived from reference [4.24]

Table C7 : Moment-Rotation Data : Mann - Test C1

Moment (kips.ft)	Rotation (Radians)
0.000	0.00000
4.050	0.00010
8.100	0.00020
12.417	0.00040
16.742	0.00060
21.058	0.00080
25.742	0.00100
30.425	0.00120
35.100	0.00140
39.058	0.00163
43.025	0.00187
46.983	0.00210
52.383	0.00250
57.783	0.00290
63.183	0.00345
68.583	0.00400
72.183	0.00450
75.783	0.00500
79.383	0.00550
82.983	0.00593
86.583	0.00637
90.183	0.00680
91.625	0.00720
93.067	0.00760
94.500	0.00800
96.667	0.00853
98.825	0.00907
100.983	0.00960
102.425	0.01000
103.867	0.01040
105.300	0.01080
106.925	0.01130
108.542	0.01180

Table C8 : Moment-Rotation Data : Mann - Test C2
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Mann, A.P. [4.25]
Test identification:	Test C2

Beam size :	15 × 6 UB 40
Column size :	10 × 10 × UC 60
End-plate :	19.25× 7.00 × 0.75 in.

Major parameters :	Ct=1.750in	Pt=4.500in	Pit=0.000in
	Cc+Pc=3.000in	Pc= -	Pic=0.000in
	Lb=14.500in	G=3.500in	Pi=10.000in

Stiffeners :	No
Beam fastening :	1/16 in. butt welds
Column fastening :	6 × 1 in. diameter HSFG bolts
Hole type :	1.25 in. oversize holes

Measured Material Properties:

	Yield Stress (Ton/in^2)	Ultimate Stress (Ton/in^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : Tension bolts preloaded

Failure moment: 306.268 kips.ft
Failure mode : Weld fracture

Remarks:
1) End-plate extended on tension side only
2) 7 in. × 3 in. shear plate welded to column to support end-plate
3) 1/2 in. thick half length column web stiffener used in compression region
4) Moment -rotation data derived from reference [4.24]

Table C8 : Moment-Rotation Data : Mann - Test C2

Moment (kips.ft)	Rotation (Radians)
0.000	0.00000
10.083	0.00020
20.017	0.00040
28.133	0.00070
36.250	0.00100
45.450	0.00125
54.650	0.00150
64.925	0.00180
75.208	0.00210
84.950	0.00240
94.683	0.00270
106.050	0.00305
117.408	0.00340
126.608	0.00370
135.808	0.00400
145.008	0.00430
154.200	0.00400
163.942	0.00490
173.683	0.00520
182.875	0.00055
192.075	0.00590
206.142	0.00650
218.050	0.00710
227.783	0.00780
239.692	0.00900
249.433	0.01010
254.842	0.01100
258.083	0.01210
262.417	0.01340
264.583	0.01480
268.908	0.01630
272.158	0.01740
273.233	0.01880
274.317	0.02000
275.400	0.02140
277.567	0.02250
278.650	0.02380
279.725	0.02490
280.808	0.02620
284.058	0.02750
282.975	0.02850
284.058	0.02990
286.225	0.03150
287.300	0.03300
289.467	0.03440

Table C9 : Moment-Rotation Data : Mann - Test C3
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Mann, A.P. [4.25]
Test identification:	Test C3

Beam size :	15 × 6 UB 40
Column size :	10 × 10 × UC 60
End-plate :	19.25× 7.00 × 0.75 in.

Major parameters :	Ct=1.750in	Pt=4.500in	Pit=0.000in
	Cc+Pc=3.000in	Pc= -	Pic=0.000in
	Lb=14.500in	G=3.500in	Pi=10.000in

Stiffeners :	No
Beam fastening :	1/16 in. butt welds
Column fastening :	6 × 1 in. diameter HSFG bolts
Hole type :	1.25 in. oversize holes

Measured Material Properties:

	Yield Stress (Ton/in^2)	Ultimate Stress (Ton/in^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : Tension bolts preloaded

Failure moment: 294.690 kips.ft
Failure mode : Weld fracture

Remarks:
1) End-plate extended on tension side only
2) 7 in. × 3 in. shear plate welded to column to support end-plate
3) 1/2 in. thick half length column web stiffener used in compression region
4) Moment-rotation data derived from reference [4.24]

Table C9 : Moment-Rotation Data : Mann - Test C3

Moment (kips.ft)	Rotation (Radians)
0.000	0.00000
8.383	0.00015
16.775	0.00030
30.300	0.00045
43.825	0.00060
54.108	0.00075
64.383	0.00090
74.667	0.00110
84.950	0.00130
94.142	0.00145
103.342	0.00160
116.300	0.00180
129.317	0.00200
140.133	0.00220
150.958	0.00240
160.158	0.00265
169.350	0.00290
179.092	0.00320
188.833	0.00350
199.108	0.00390
209.392	0.00430
216.967	0.00460
224.542	0.00490
237.525	0.00580
245.100	0.00625
252.675	0.00670
262.417	0.00760
271.075	0.00890
277.567	0.00990
281.892	0.01120
284.058	0.01220
286.225	0.01320
288.383	0.01430
289.467	0.01540
290.550	0.01660
293.800	0.01790
293.800	0.01880
292.717	0.02020
292.717	0.02190
292.717	0.02320
290.550	0.02450
290.550	0.02590
292.717	0.02750
292.717	0.02880
290.550	0.02990
291.092	0.03125
291.633	0.03260

Table C10 : Moment-Rotation Data : Mann - Test C4
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Mann, A.P. [4.25]
Test identification:	Test C4

Beam size :	15 × 6 UB 40
Column size :	10 × 10 × UC 60
End-plate :	19.25× 7.00 × 1.0 in.

Major parameters :	Ct=1.750in	Pt=4.500in	Pit=0.000in
	Cc+Pc=3.000in	Pc= -	Pic=0.000in
	Lb=14.500in	G=3.500in	Pi=10.000in

Stiffeners :	No
Beam fastening :	1/16 in. butt welds
Column fastening :	6 × 1.0 in. diameter HSFG bolts
Hole type :	1.25 in. oversize holes

Measured Material Properties:

	Yield Stress (Ton/in^2)	Ultimate Stress (Ton/in^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 70 tons.

Failure moment: 361.150 kips.ft
Failure mode : Excessive deformation

Remarks:
1) End-plate extended on tension side only
2) 7 in. × 3 in. shear plate welded to column to support end-plate
3) 0.5 in. thick half length column web stiffener used in compression region
4) Moment -rotation data derived from reference [4.24]

Table C10 : Moment-Rotation Data : Mann - Test C4

Moment (kips.ft)	Rotation (Radians)
0.000	0.00000
10.533	0.00040
21.058	0.00080
34.025	0.00125
46.983	0.00170
56.700	0.00205
66.425	0.00240
78.300	0.00280
90.183	0.00320
102.600	0.00365
115.025	0.00410
124.742	0.00445
134.467	0.00480
146.342	0.00520
158.225	0.00560
169.567	0.00595
180.908	0.00630
192.783	0.00670
204.667	0.00710
214.925	0.00740
225.192	0.00770
234.908	0.00815
244.625	0.00860
256.508	0.00920
266.225	0.01000
274.867	0.01130
279.192	0.01310
281.892	0.01435
284.592	0.01560
287.833	0.01675
291.067	0.01790
291.067	0.01910
291.067	0.02030
292.150	0.02140
293.233	0.02250
295.933	0.02445
298.633	0.02640
299.175	0.02765
299.708	0.02890
301.333	0.03005
302.950	0.03120
304.575	0.03255
306.192	0.03390
308.350	0.03535
310.508	0.03680

Table C11 : Moment-Rotation Data : Mann - Test C5
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Mann, A.P. [4.25]
Test identification:	Test C5

Beam size :	15 × 6 UB 40
Column size :	10 × 10 × UC 60
End-plate :	19.25× 7.00 × 1.0 in.

Major parameters :	Ct=1.750in	Pt=4.500in	Pit=0.000in
	Cc+Pc=3.000in	Pc= -	Pic=0.000in
	Lb=14.500in	G=3.500in	Pi=10.000in

Stiffeners :	No
Beam fastening :	1/16 in. butt welds
Column fastening :	6 × 1.0 in. diameter HSFG bolts
Hole type :	1.25 in. oversize holes

Measured Material Properties:

	Yield Stress (Ton/in^2)	Ultimate Stress (Ton/in^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : Tension bolts preloaded

Failure moment: 361.150 kips.ft
Failure mode : Excessive deformation

Remarks:
1) End-plate extended on tension side only
2) 7 in. × 3 in. shear plate welded to column to support end-plate
3) 0.5 in. thick half length column web stiffener used in compression region
4) Moment -rotation data derived from reference [4.24]

Table C11 : Moment-Rotation Data : Mann - Test C5

Moment (kips.ft)	Rotation (Radians)
0.000	0.00000
13.742	0.00025
27.492	0.00050
37.192	0.00075
46.892	0.00100
64.142	0.00150
80.308	0.00190
95.400	0.00220
106.175	0.00250
116.958	0.00280
128.817	0.00310
140.675	0.00340
150.917	0.00370
161.158	0.00400
172.475	0.00435
183.792	0.00470
192.958	0.00495
202.117	0.00520
216.133	0.00580
229.067	0.00640
238.767	0.00720
249.550	0.00820
257.092	0.00910
263.558	0.01040
268.950	0.01150
273.267	0.01290
277.575	0.01420
284.042	0.01570
287.275	0.01700
290.508	0.01830
294.825	0.01930
298.058	0.02120
302.367	0.02260
304.525	0.02390
307.758	0.02550
314.225	0.02680
316.383	0.02830
320.692	0.02970
323.925	0.03075
327.158	0.03180
329.317	0.03310
335.783	0.03500
337.942	0.03630
340.842	0.03770
344.408	0.03890

Table C12 : Moment-Rotation Data : Mann - Test C6
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Mann, A.P. [4.25]
Test identification:	Test C6

Beam size :	18 × 6 UB 55
Column size :	10 × 10 × UC 89
End-plate :	24.0× 10.0 × 1.125 in.

Major parameters :	Ct=1.750in	Pt=6.000in	Pit=0.000in
	Cc+Pc=3.000in	Pc= -	Pic=0.000in
	Lb=19.250in	G=6.000in	Pi=13.250in

Stiffeners :	0.5 in. thick column web stiffeners opposite both beam flanges
Beam fastening :	1/16 in. butt welds
Column fastening :	6 × 1.125 in. diameter HSFG bolts
Hole type :	

Measured Material Properties:

	Yield Stress (Ton/in^2)	Ultimate Stress (Ton/in^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : Tension bolts preloaded

Failure moment: 439.50 kips.ft
Failure mode : Weld fracture

Remarks:
1) End-plate extended on tension side only
2) 7 in. × 3 in. shear plate welded to column to support end-plate
3) Moment-rotation data derived from reference [4.24]

Table C12 : Moment-Rotation Data : Mann - Test C6

Moment (kips.ft)	Rotation (Radians)
0.000	0.00000
17.300	0.00015
34.592	0.00030
47.567	0.00050
60.542	0.00070
80.000	0.00095
99.458	0.00120
115.133	0.00140
130.808	0.00160
144.867	0.00180
158.917	0.00200
176.758	0.00225
194.592	0.00250
210.267	0.00270
225.950	0.00290
241.625	0.00310
257.300	0.00330
269.192	0.00350
281.083	0.00370
295.133	0.00385
309.192	0.00400
324.867	0.00420
340.542	0.00440
361.083	0.00480
376.217	0.00520
391.350	0.00610
404.325	0.00710
411.892	0.00840
416.217	0.00990
421.625	0.01150
423.783	0.01320
428.108	0.01480
431.350	0.01610
434.592	0.01760
435.675	0.01910
437.842	0.02090
438.917	0.02220
440.000	0.02370
441.083	0.02570
444.325	0.02720
445.408	0.02890
446.483	0.03020
448.650	0.03130
449.733	0.03260
450.808	0.03390
451.892	0.03510
454.050	0.03630

Table C13 : Moment-Rotation Data : Bailey - Test A1-L
Connection Type : Extended end-plate connection to unstiffened column

Tested by : Bailey, J.R. [4.26]
Test identification: Test A1-Left

Beam size : 320 mm × 127 mm × 48 kg/m
Column size : 216 mm × 216 mm × 71 kg/m
End-plate : 422 × 203 × 25.4 mm

Major parameters :			
	Ct= -	Pt=4.500in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners : NO
Beam fastening : Weld of 17.4 mm for upper flange and 11.1 mm for remainder
Column fastening : 4 × 22.2 mm diameter HSFG bolts in tension and 4 × 25.4 mm diameter HSFG bolts in compression
Hole type : Drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 258.75 kNm
Failure mode : Column web buckling

Remarks:
1) End-plate extended on tension side only
2) Two 22.2 mm diameter bolts used on either side of tension flange and four 25.4 mm bolts in compression
3) High moment loading
4) Only moment-deflection data available , moment-rotation data derived from reference [4.24,4.24]

Table C13 : Moment-Rotation Data : Bailey - Test A1-L

Moment (kNm)	Rotation (Radians)
0.00	0.00000
4.07	0.00030
8.14	0.00060
12.21	0.00090
16.27	0.00120
21.38	0.00140
26.49	0.00160
31.60	0.00180
39.26	0.00215
46.92	0.00250
54.58	0.00285
62.24	0.00320
72.30	0.00365
82.35	0.00410
90.00	0.00445
97.67	0.00480
105.81	0.00520
113.95	0.00560
123.52	0.00605
133.10	0.00650
139.80	0.00685
146.51	0.00720
153.68	0.00765
160.87	0.00810
172.36	0.00880
182.89	0.00920
192.47	0.01000
203.96	0.01120
212.57	0.01250
220.23	0.01350
226.94	0.01490
232.68	0.01660
238.43	0.01810
240.82	0.01905
243.22	0.02000
246.09	0.02160
247.52	0.02250
248.97	0.02340
249.91	0.02430
250.87	0.02520
251.83	0.02650
252.79	0.02790
252.79	0.02960
253.27	0.03070
253.75	0.03180

Table C14 : Moment-Rotation Data : Bailey - Test A1-R
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test A1-Right

Beam size :	320 mm × 127 mm × 48 kg/m
Column size :	216 mm × 216 mm × 71 kg/m
End-plate :	422 × 203 × 25.4 mm

Major parameters :	Ct= -	Pt=4.500in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners :	NO
Beam fastening :	Weld of 17.4 mm for upper flange and 11.1 mm for remainder
Column fastening :	4 × 22.2 mm diameter HSFG bolts in tension and 4 × 25.4 mm diameter HSFG bolts in compression
Hole type :	Drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 258.75 kNm
Failure mode : Column web buckling

Remarks:
1) End-plate extended on tension side only
2) Two 22.2 mm diameter bolts used on either side of tension flange and four 25.4 mm bolts in compression
3) High moment loading
4) Only moment-deflection data available , moment-rotation data derived from reference [4.23,4.24]

Table C14 : Moment-Rotation Data : Bailey - Test A1-R

Moment (kNm)	Rotation (Radians)
0.00	0.00000
9.03	0.00020
18.06	0.00040
29.45	0.00080
36.59	0.00100
43.72	0.00120
51.80	0.00145
59.87	0.00170
72.23	0.00220
79.36	0.00235
86.48	0.00250
96.93	0.00270
109.29	0.00320
115.95	0.00340
122.59	0.00360
129.72	0.00380
136.85	0.00400
144.45	0.00420
152.06	0.00440
160.61	0.00480
172.00	0.00530
182.46	0.00590
194.82	0.00680
203.37	0.00780
211.93	0.00900
217.63	0.01020
222.38	0.01144
229.04	0.01270
232.84	0.01370
235.21	0.01450
237.59	0.01530
241.40	0.01660
245.19	0.01800
247.10	0.01940
249.00	0.02090
250.41	0.02180
251.85	0.02270
252.79	0.02410
253.74	0.02540
253.74	0.02650
254.70	0.02770
255.65	0.02880
255.65	0.02960
255.65	0.03060
255.65	0.03140

Table C15 : Moment-Rotation Data : Bailey - Test A2-L Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test A2-Left

Beam size :	254 mm × 101.9 mm × 25 kg/m
Column size :	216 mm × 216 mm × 59 kg/m
End-plate :	422 × 165 × 19.0 mm

Major parameters :	Ct= -	Pt=4.375in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners :	NO
Beam fastening :	Weld of 11.1 mm for upper flange and 9.5 mm for remainder
Column fastening :	4 × 16 mm diameter HSFG bolts in tension and 4 × 22.2 mm diameter HSFG bolts in compression
Hole type :	Drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

pretension force of bolts : -

Failure moment: 143.3 kNm
Failure mode : Plasticity

Remarks:
1) End-plate extended on tension side only
2) Two 16 mm diameter bolts used on either side of tension flange and four 22.2 mm bolts in compression
3) High moment loading
4) Only moment-deflection data available , moment-rotation data derived from references [4.23,4.24]

Table C15 : Moment-Rotation Data : Bailey - Test A2-L

Moment (kNm)	Rotation (Radians)
0.00	0.00000
1.00	0.00010
2.02	0.00020
3.02	0.00030
4.03	0.00040
5.04	0.00050
6.05	0.00060
9.07	0.00090
13.17	0.00110
17.27	0.00130
23.42	0.00160
27.24	0.00180
31.06	0.00200
36.79	0.00220
42.53	0.00240
47.31	0.00255
52.09	0.00270
55.91	0.00300
59.73	0.00330
64.51	0.00365
69.29	0.00400
73.12	0.00420
76.93	0.00440
81.72	0.00460
86.49	0.00480
90.32	0.00510
94.14	0.00540
97.97	0.00570
101.78	0.0060
107.52	0.00690
111.34	0.00730
115.17	0.00770
119.94	0.00890
123.77	0.00980
126.55	0.01230
129.98	0.01310
131.42	0.01390
132.85	0.01465
134.28	0.01540
135.72	0.01610
137.15	0.01680
137.15	0.01755
137.15	0.01830
137.63	0.01915
138.10	0.02000

Table C16 : Moment-Rotation Data : Bailey - Test A2-R Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test A2-Right

Beam size :	254 mm × 101.9 mm × 25 kg/m
Column size :	216 mm × 216 mm × 59 kg/m
End-plate :	422 × 165 × 19.0 mm

Major parameters :	Ct= -	Pt=4.375in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners :	NO
Beam fastening :	Weld of 11.1 mm for upper flange and 9.5 mm for remainder
Column fastening :	4 × 16 mm diameter HSFG bolts in tension and 4 × 22.2 mm diameter HSFG bolts in compression
Hole type :	Drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 143.3 kNm
Failure mode : Plasticity

Remarks:
1) End-plate extended on tension side only
2) Two 16 mm diameter bolts used on either side of tension flange and four 22.2 mm bolts in compression
3) High moment loading
4) Only moment-deflection data available , moment-rotation data derived from reference [4.23,4.24]

Table C16 : Moment-Rotation Data : Bailey - Test A2-R

Moment (kNm)	Rotation (Radians)
0.00	0.00000
4.62	0.00030
9.24	0.00060
13.86	0.00090
19.59	0.00125
25.32	0.00160
30.58	0.00175
35.84	0.00190
40.62	0.00210
45.40	0.00230
51.13	0.00235
56.87	0.00240
60.21	0.00265
63.56	0.00290
67.37	0.00315
71.20	0.00340
74.55	0.00360
77.89	0.00380
82.19	0.00410
86.49	0.00440
92.22	0.00520
95.58	0.00570
98.92	0.00620
102.27	0.00665
105.61	0.00710
110.38	0.00800
115.17	0.00920
119.94	0.01060
122.81	0.01190
123.77	0.01265
124.73	0.01340
125.67	0.01415
126.67	0.01415
126.63	0.01490
128.55	0.01620
129.03	0.01720
129.50	0.01820
129.50	0.01895
129.50	0.01970
129.98	0.02065
130.46	0.02160
130.46	0.02240
130.46	0.02320
130.93	0.02400
131.42	0.02480

Table C17 : Moment-Rotation Data : Bailey - Test A3-L
Connection Type : Extended end-plate connection to unstiffened column

Tested by : Bailey, J.R. [4.26]
Test identification: Test A3-Left

Beam size : 254 mm × 101.9 mm × 28 kg/m
Column size : 216 mm × 216 mm × 59 kg/m
End-plate : 422 × 171 × 19.0 mm

Major parameters :			
	Ct= -	Pt=4.375in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners : NO
Beam fastening : Weld of 12.7 mm for upper flange and 9.5 mm for remainder
Column fastening : 4 × 19.0 mm diameter HSFG bolts in tension and 4 × 22.2 mm diameter HSFG bolts in compression
Hole type : Drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 157.9 kNm
Failure mode : Plasticity

Remarks:
1) End-plate extended on tension side only
2) Two 19.0 mm diameter bolts used on either side of tension flange and four 22.2 mm bolts in compression
3) High moment loading
4) Only moment-deflection data available , moment-rotation data derived from reference [4.23,4.24]

Table C17 : Moment-Rotation Data : Bailey - Test A3-L

Moment (kNm)	Rotation (Radians)
0.00	0.00000
2.20	0.00020
4.41	0.00040
6.62	0.00060
8.82	0.00080
11.04	0.00100
17.75	0.00135
24.47	0.00170
30.70	0.00200
36.98	0.00230
42.25	0.00255
47.54	0.00280
51.86	0.00310
56.18	0.00340
61.63	0.00363
67.07	0.00387
72.51	0.00410
78.76	0.00445
85.00	0.00480
90.28	0.00510
95.60	0.00540
101.81	0.00585
108.06	0.00630
114.29	0.00665
120.54	0.00700
125.82	0.00755
131.10	0.00810
134.95	0.00870
138.80	0.00930
141.67	0.01015
144.55	0.01100
146.94	0.01160
149.35	0.01220
150.78	0.01295
152.23	0.01370
153.20	0.01455
154.15	0.01540
155.11	0.01620
156.07	0.01700
156.39	0.01780
156.72	0.01860
157.03	0.01940
159.56	0.02020
156.07	0.02100

Table C18 : Moment-Rotation Data : Bailey - Test A3-R Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test A3-Right

Beam size :	254 mm × 101.9 mm × 28 kg/m
Column size :	216 mm × 216 mm × 59 kg/m
End-plate :	422 × 171 × 19.0 mm

Major parameters :	Ct= -	Pt=4.375in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners :	NO
Beam fastening :	Weld of 12.7 mm for upper flange and 9.5 mm for remainder
Column fastening :	4 × 19.0 mm diameter HSFG bolts in tension and 4 × 22.2 mm diameter HSFG bolts in compression
Hole type :	Drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 164.0 kNm
Failure mode : Plasticity

Remarks:
1) End-plate extended on tension side only
2) Two 19.0 mm diameter bolts used on either side of tension flange and four 22.2 mm bolts in compression
3) High moment loading
4) Only moment-deflection data available , moment-rotation data derived from reference [4.23,4.24]

Table C18 : Moment-Rotation Data : Bailey - Test A3-R

Moment (kNm)	Rotation (Radians)
0.00	0.00000
5.58	0.00023
11.16	0.00047
16.74	0.00070
22.48	0.00100
28.22	0.00130
33.96	0.00160
39.71	0.00190
45.45	0.00215
51.19	0.00240
56.93	0.00255
62.67	0.00270
67.46	0.00295
72.24	0.00320
77.50	0.00350
82.77	0.00380
88.02	0.00405
93.29	0.00430
98.55	0.00450
103.82	0.00470
108.60	0.00500
113.38	0.00530
119.60	0.00570
125.82	0.00610
133.48	0.00730
141.14	0.00840
143.52	0.00905
145.92	0.00970
150.70	0.01080
152.13	0.01145
153.56	0.01210
154.52	0.01295
155.48	0.01380
156.44	0.01455
157.40	0.01530
158.83	0.01595
160.26	0.01660
160.75	0.01755
161.22	0.01850
160.75	0.01950
160.26	0.02050
161.22	0.02135
162.18	0.02220
161.86	0.02303
161.54	0.02387

Table C19 : Moment-Rotation Data : Bailey - Test B4-L Connection Type : Extended end-plate connection to stiffened column

Tested by : Bailey, J.R. [4.26]
Test identification: Test B4-Left

Beam size : 320 mm × 127 mm × 48 kg/m
Column size : 216 mm × 216 mm × 59 kg/m
End-plate : 422 × 177.5 × 31.8 mm

Major parameters :			
	Ct= -	Pt=4.500in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners : 7.9 mm column web stiffeners opposite both beam flanges
Beam fastening : Weld of 16.0 mm for upper flange and [B 4.8 mm for remainder
Column fastening : 4 × 22.2 mm diameter HSFG bolts in tension and 4 × 19.0 mm diameter HSFG bolts in compression
Hole type : Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 267.3 kNm
Failure mode : Plasticity in beams and column flanges

Remarks:
1) End-plate extended on tension side only
2) Two 22.2 mm diameter bolts used on either side of tension flange and four 19.0 mm bolts in compression
3) High moment loading
4) Only moment-deflection data available , moment-rotation data derived from reference [4.23,4.24]

Table C19 : Moment-Rotation Data : Bailey - Test B4-L

Moment (kNm)	Rotation (Radians)
0.00	0.00000
7.90	0.00025
15.78	0.00050
15.78	0.00040
23.92	0.00060
32.05	0.00080
42.10	0.00100
52.15	0.00120
59.33	0.00150
66.50	0.00180
73.68	0.00210
80.85	0.00240
95.21	0.00260
104.30	0.00290
113.38	0.00320
121.04	0.00330
128.70	0.00340
137.78	0.00370
146.87	0.00400
159.31	0.00420
166.96	0.00490
179.40	0.00550
189.93	0.00660
198.54	0.00770
206.19	0.00920
216.72	0.01080
224.37	0.01280
231.07	0.01480
234.90	0.01670
240.64	0.01880
242.55	0.02040
246.38	0.02230
248.30	0.02390
250.21	0.02580
252.12	0.02790
255.00	0.02970
255.95	0.03110
257.86	0.03270
258.82	0.03460
259.78	0.03630
260.74	0.03830
261.17	0.04030
261.17	0.04240

Table C20 : Moment-Rotation Data : Bailey - Test B4-R Connection Type : Extended end-plate connection to stiffened column

Tested by : Bailey, J.R. [4.26]
Test identification: Test B4-Right

Beam size : 320 mm × 127 mm × 48 kg/m
Column size : 216 mm × 216 mm × 59 kg/m
End-plate : 422 × 177.5 × 31.8 mm

Major parameters :	Ct= -	Pt=4.500in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners : 7.9 mm column web stiffeners ooposit both beam flanges
Beam fastening : Weld of 16.0 mm for upper flange and 4.8 mm for remainder
Column fastening : 4 × 22.2 mm diameter IISFG bolts in tension and 4 × 19.0 mm diameter IISFG bolts in compression
Hole type : Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 267.3 kNm
Failure mode : Plasticity in beams and column flanges [B

Remarks:
1) End-plate extended on tension side only
2) Two 22.2 mm diameter bolts used on either side of tension flange and four 19.0 mm bolts in compression
3) High moment loading
4) Only moment-deflection data available , moment-rotation data derived from reference [4.23,4.24]

Table C20 : Moment-Rotation Data : Bailey - Test B4-R

Moment (kNm)	Rotation (Radians)
0.00	0.00000
11.94	0.00030
23.87	0.00060
31.51	0.00065
39.16	0.00070
47.27	0.00095
55.40	0.00120
64.47	0.00135
73.54	0.00150
80.70	0.00165
87.87	0.00180
95.50	0.00205
103.14	0.00230
109.83	0.00235
116.50	0.00240
124.63	0.00260
132.74	0.00280
145.16	0.00320
156.63	0.00350
164.26	0.00410
174.77	0.00520
187.19	0.00660
196.73	0.00770
206.28	0.00900
212.00	0.01040
219.66	0.01190
222.52	0.01320
226.34	0.01460
229.21	0.01620
233.03	0.01800
236.84	0.01980
239.71	0.02180
242.57	0.02360
246.39	0.02560
248.30	0.02750
251.17	0.02980
253.08	0.03150
254.04	0.03320
255.95	0.03530
256.90	0.03700
257.86	0.03900
257.86	0.04090
258.81	0.04270
259.77	0.04420
260.72	0.04570

Table C21 : Moment-Rotation Data : Bailey - Test B5-L Connection Type : Extended end-plate connection to stiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test B5-Left

Beam size :	366 mm × 127 mm × 39 kg/m
Column size :	216 mm × 216 mm × 59 kg/m
End-plate :	422 × 177.5 × 25.4 mm

Major parameters :	Ct= -	Pt=4.250in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners :	7.9 mm column web stiffeners opposite both beam flanges
Beam fastening :	Weld of 12.7 mm for upper flange and 4.8 mm for remainder
Column fastening :	4 × 19.0 mm diameter HSFG bolts in tension and 4 × 19.0 mm diameter HSFG bolts in compression
Hole type :	Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 230.8 kNm
Failure mode : Beam flange buckling

Remarks:
1) End-plate extended on tension side only
2) Two 19.0 mm diameter bolts used on either side of tension flange and four 19.0 mm bolts in compression
3) High moment loading
4) Only moment-deflection data available , moment-rotation data derived from reference [4.23,4.24]

Table C21 : Moment-Rotation Data : Bailey - Test B5-L

Moment (kNm)	Rotation (Radians)
0.00	0.00000
8.63	0.00035
17.25	0.00070
26.36	0.00095
35.46	0.00120
43.14	0.00145
50.81	0.00170
60.39	0.00200
69.98	0.00230
78.13	0.00260
86.27	0.00290
94.43	0.00310
102.58	0.00330
112.16	0.00360
121.75	0.00390
128.93	0.00420
136.12	0.00450
141.88	0.00485
147.63	0.00520
155.30	0.00590
162.96	0.00660
168.72	0.00720
174.47	0.00780
185.02	0.00930
193.64	0.01080
201.31	0.01220
206.10	0.01380
209.94	0.01550
213.77	0.01760
217.61	0.01960
219.52	0.02085
221.44	0.02210
223.36	0.02410
223.84	0.02520
224.32	0.02630
225.27	0.02820
225.27	0.03010
225.27	0.03120
225.27	0.03230
227.20	0.03420
227.20	0.03555
227.20	0.03690
227.20	0.03860
227.20	0.04050
228.15	0.04220

Table C22 : Moment-Rotation Data : Bailey - Test B5-R Connection Type : Extended end-plate connection to stiffened column

Tested by : Bailey, J.R. [4.26]
Test identification: Test B5-Right

Beam size : 366 mm × 127 mm × 39 kg/m
Column size : 216 mm × 216 mm × 59 kg/m
End-plate : 422 × 177.5 × 25.4 mm

Major parameters :			
	Ct= -	Pt=4.250in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners : 7.9 mm column web stiffeners opposite both beam flanges

Beam fastening : Weld of 12.7 mm for upper flange and 4.8 mm for remainder

Column fastening : 4 × 19.0 mm diameter HSFG bolts in tension and 4 × 19.0 mm diameter HSFG bolts in compression

Hole type : Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 230.8 kNm
Failure mode : Beam flange buckling

Remarks:
1) End-plate extended on tension side only
2) Two 19.0 mm diameter bolts used on either side of tension flange and four 19.0 mm bolts in compression
3) High moment loading
4) Only moment-deflection data available , moment-rotation data derived from reference [4.23,4.24]

Table C22 : Moment-Rotation Data : Bailey - Test B5-R

Moment (kNm)	Rotation (Radians)
0.00	0.00000
8.17	0.00013
16.32	0.00027
24.50	0.00040
31.69	0.00060
38.90	0.00080
47.54	0.00100
56.18	0.00120
65.79	0.00140
75.40	0.00160
81.15	0.00170
86.92	0.00180
94.60	0.00205
102.30	0.00230
109.98	0.00225
117.66	0.00220
127.26	0.00250
136.87	0.00280
143.10	0.00030
149.35	0.00320
156.07	0.00350
162.80	0.00380
169.52	0.00450
177.20	0.00550
185.84	0.00640
192.57	0.00730
197.37	0.00820
202.18	0.00930
206.01	0.01060
208.90	0.01210
213.70	0.01330
216.58	0.01490
217.55	0.01650
220.43	0.01780
222.35	0.01990
223.31	0.02180
224.27	0.02360
225.23	0.02540
225.23	0.02730
225.23	0.02920
226.19	0.03110
226.19	0.03300
226.19	0.03440
226.19	0.03600
227.15	0.03800
227.15	0.03920
228.11	0.04070

Table C23 : Moment-Rotation Data : Bailey - Test B6-L Connection Type : Extended end-plate connection to stiffened column

Tested by : Bailey, J.R. [4.26]
Test identification: Test B6-Left

Beam size : 366 mm × 171 mm × 67 kg/m
Column size : 216 mm × 216 mm × 59 kg/m
End-plate : 422 × 203 × 34.9 mm

Major parameters :			
	Ct= -	Pt=4.625in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners : 7.9 mm column web stiffeners opposite both beam flanges
Beam fastening : Weld of 17.4 mm for upper flange and 4.8 mm for remainder
Column fastening : 4 × 25.4 mm diameter HSFG bolts in tension and 4 × 28.6 mm diameter HSFG bolts in compression
Hole type : Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 427.6 kNm
Failure mode : Column web buckling

Remarks:
1) End-plate extended on tension side only
2) Two 25.4 mm diameter bolts used on either side of tension flange and four 28.6 mm bolts in compression
3) High moment loading
4) Only moment-deflection data available , moment-rotation data derived from reference [4.23,4.24]

Table C23 : Moment-Rotation Data : Bailey - Test B6-L

Moment (kNm)	Rotation (Radians)
0.00	0.00000
17.32	0.00065
34.64	0.00130
54.85	0.00175
75.05	0.00220
89.49	0.00250
103.93	0.00280
117.40	0.00315
130.87	0.00350
145.31	0.00380
159.73	0.00410
173.21	0.00450
186.68	0.00480
198.23	0.00505
209.77	0.00530
220.36	0.00565
230.95	0.00600
247.30	0.00670
263.65	0.00740
275.21	0.00780
286.76	0.00820
304.08	0.00950
314.67	0.01025
325.25	0.01100
338.72	0.01230
354.11	0.01380
358.93	0.01495
363.76	0.01610
373.37	0.01790
377.21	0.01960
381.06	0.02110
388.76	0.02270
392.60	0.02450
394.52	0.02630
397.41	0.02750
400.29	0.02870
402.25	0.03060
403.21	0.03180
404.17	0.03300
409.94	0.03510
410.90	0.03625
411.86	0.03740
417.63	0.03940
423.40	0.04140
423.40	0.04330

Table C24 : Moment-Rotation Data : Bailey - Test B6-R Connection Type : Extended end-plate connection to stiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test B6-Right

Beam size :	366 mm × 171 mm × 67 kg/m
Column size :	216 mm × 216 mm × 59 kg/m
End-plate :	422 × 203 × 34.9 mm

Major parameters :	Ct= -	Pt=4.625in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners :	7.9 mm column web stiffeners opposite both beam flanges
Beam fastening :	Weld of 17.4 mm for upper flange and 4.8 mm for remainder
Column fastening :	4 × 25.4 mm diameter HSFG bolts in tension and 4 × 28.6 mm diameter HSFG bolts in compression
Hole type :	Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 433.7 kNm
Failure mode : Column web buckling

Remarks:
1) End-plate extended on tension side only
2) Two 25.4 mm diameter bolts used on either side of tension flange and four 28.6 mm bolts in compression
3) High moment loading
4) Only moment-deflection data available , moment-rotation data derived from reference [4.23,4.24]

Table C24 : Moment-Rotation Data : Bailey - Test B6-R

Moment (kNm)	Rotation (Radians)
0.00	0.00000
14.70	0.00033
29.40	0.00067
44.10	0.00100
64.23	0.00120
84.36	0.00140
97.77	0.00180
111.20	0.00220
129.41	0.00260
147.63	0.00300
166.81	0.00335
185.98	0.00370
198.44	0.00405
210.90	0.00440
223.36	0.00465
235.83	0.00490
252.12	0.00540
268.41	0.00590
281.84	0.00655
295.27	0.00720
306.77	0.00790
318.27	0.00860
335.53	0.00970
354.69	0.01110
366.21	0.01260
377.70	0.01440
381.54	0.01620
389.21	0.01780
398.80	0.02000
400.70	0.02210
404.54	0.02410
410.29	0.02570
410.29	0.02790
414.13	0.03020
416.03	0.03200
419.88	0.03410
420.85	0.03530
421.80	0.03650
421.80	0.03820
425.62	0.03980
423.70	0.04100
421.80	0.04280
423.70	0.04420
423.70	0.04570
425.62	0.04650

Table C25 : Moment-Rotation Data : Bailey - Test B7-L Connection Type : Extended end-plate connection to stiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test B7-Left

Beam size :	320 mm × 165 mm × 54 kg/m
Column size :	216 mm × 216 mm × 59 kg/m
End-plate :	422 × 190.5 × 31.8 mm

Major parameters :	Ct= -	Pt=4.500in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners :	7.9 mm column web stiffeners opposite both beam flanges
Beam fastening :	Weld of 17.4 mm for upper flange and 11.1 mm for remainder
Column fastening :	4 × 25.4 mm diameter HSFG bolts in tension and 4 × 19.0 mm diameter HSFG bolts in compression
Hole type :	Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 291.5 kNm
Failure mode : Plasticity in beams and column flanges

Remarks:
1) End-plate extended on tension side only
2) Two 25.4 mm diameter bolts used on either side of tension flange and four 19.0 mm bolts in compression
3) High moment loading
4) Only moment-deflection data available , moment-rotation data derived from references [4.23,4.24]

Table C25 : Moment-Rotation Data : Bailey - Test B7-L

Moment (kNm)	Rotation (Radians)
0.00	0.00000
12.65	0.00050
25.31	0.00100
36.75	0.00137
48.21	0.00173
59.65	0.00210
72.31	0.00255
84.96	0.00300
95.81	0.00335
106.65	0.00370
118.40	0.00390
130.15	0.00410
141.00	0.00435
151.85	0.00460
161.79	0.00510
171.73	0.00560
181.67	0.00605
191.62	0.00650
201.56	0.00720
211.50	0.00790
218.73	0.00865
225.96	0.00940
235.23	0.01040
244.04	0.01140
254.89	0.01350
260.12	0.01680
267.54	0.01890
269.35	0.02110
272.96	0.02350
276.58	0.02570
277.48	0.02695
278.39	0.02820
280.19	0.03050
283.81	0.03290
283.81	0.03480
282.90	0.03625
282.00	0.03770
285.62	0.03970
285.62	0.04150
289.23	0.04340
287.42	0.04540
287.42	0.04690
289.23	0.04830

Table C26 : Moment-Rotation Data : Bailey - Test B7-R
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test B7-Right

Beam size :	320 mm × 165 mm × 54 kg/m
Column size :	216 mm × 216 mm × 59 kg/m
End-plate :	422 × 190.5 × 31.8 mm

Major parameters :	Ct= -	Pt=4.500in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners :	7.9 mm column web stiffeners opposite both beam flanges
Beam fastening :	Weld of 17.4 mm for upper flange and 11.1 mm for remainder
Column fastening :	4 × 25.4 mm diameter HSFG bolts in tension and 4 × 19.0 mm diameter HSFG bolts in compression
Hole type :	Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 304.9 kNm
Failure mode : Plasticity in beams and column flanges

Remarks:
1) End-plate extended on tension side only
2) Two 25.4 mm diameter bolts used on either side of tension flange and four 19.0 mm bolts in compression
3) High moment loading
4) Only moment-deflection data available , moment-rotation data derived from references [4.23,4.24]

Table C26 : Moment-Rotation Data : Bailey - Test B7-R

Moment (kNm)	Rotation (Radians)
0.00	0.00000
10.28	0.00033
20.56	0.00067
30.84	0.00100
44.45	0.00140
58.05	0.00180
70.75	0.00210
83.45	0.00240
94.34	0.00260
105.22	0.00280
116.11	0.00313
126.99	0.00347
137.88	0.00380
148.76	0.00417
159.65	0.00433
170.53	0.00490
181.41	0.00545
192.31	0.00600
205.00	0.00670
217.70	0.00740
232.21	0.00870
246.73	0.01000
254.90	0.01110
263.05	0.01220
266.68	0.01325
270.31	0.01430
274.85	0.01530
279.38	0.01630
283.00	0.01745
286.65	0.01860
287.55	0.01985
288.45	0.02110
291.18	0.02225
293.90	0.02340
295.71	0.02500
295.71	0.02610
295.71	0.02720
295.71	0.02815
295.71	0.02910
296.62	0.03040
297.52	0.03170
299.34	0.03340
299.34	0.03460
299.34	0.03580
299.34	0.03710

Table C27 : Moment-Rotation Data : Bailey - Test B8-L Connection Type : Extended end-plate connection to stiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test B8-Left

Beam size :	203 mm × 133 mm × 30 kg/m
Column size :	216 mm × 216 mm × 59 kg/m
End-plate :	422 × 165 × 19.0 mm

Major parameters :	Ct= -	Pt=4.500in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners :	7.9 mm column web stiffeners opposite both beam flanges
Beam fastening :	Weld of 11.1 mm for upper flange and 4.8 mm for remainder
Column fastening :	4 × 19.0 mm diameter HSFG bolts in tension and 4 × 12.7 mm diameter HSFG bolts in compression
Hole type :	Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 114.2 kNm
Failure mode : Plasticity in end-plate

Remarks:
1) End-plate extended on tension side only
2) Two 19.0 mm diameter bolts used on either side of tension flange and four 12.7 mm bolts in compression
3) High moment loading
4) Only moment-deflection data available , moment-rotation data derived from references [4.23,4.24]

Table C27 : Moment-Rotation Data : Bailey - Test B8-L

Moment (kNm)	Rotation (Radians)
0.00	0.00000
4.47	0.00017
8.95	0.00033
13.42	0.00050
17.58	0.00060
21.73	0.00070
25.88	0.00080
31.63	0.00095
37.39	0.00110
42.18	0.00130
46.98	0.00150
51.77	0.00158
56.56	0.00165
61.35	0.00173
66.15	0.00180
71.42	0.00205
76.70	0.00230
82.44	0.00310
88.19	0.00390
91.07	0.00440
93.94	0.00490
96.82	0.00540
99.69	0.00590
100.65	0.00650
101.62	0.00710
105.45	0.00790
106.89	0.00860
108.33	0.00930
109.29	0.01010
110.25	0.01090
110.72	0.01155
111.20	0.01220
111.84	0.01290
112.48	0.01360
113.12	0.01430
113.60	0.01500
114.77	0.01570
114.77	0.01637
115.35	0.01703
116.00	0.01770

Table C28 : Moment-Rotation Data : Bailey - Test B8-R Connection Type : Extended end-plate connection to stiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test B8-Right

Beam size :	203 mm × 133 mm × 30 kg/m
Column size :	216 mm × 216 mm × 59 kg/m
End-plate :	422 × 165 × 19.0 mm

Major parameters :	Ct= -	Pt=4.500in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners :	7.9 mm column web stiffeners opposite both beam flanges
Beam fastening :	Weld of 11.1 mm for upper flange and 4.8 mm for remainder
Column fastening :	4 × 19.0 mm diameter HSFG bolts in tension and 4 × 12.7 mm diameter HSFG bolts in compression
Hole type :	Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 109.3 kNm
Failure mode : Plasticity in end-plate

Remarks:
1) End-plate extended on tension side only
2) Two 19.0 mm diameter bolts used on either side of tension flange and four 12.7 mm bolts in compression
3) High moment loading
4) Only moment-deflection data available , moment-rotation data derived from references [4.23,4.24]

Table C28 : Moment-Rotation Data : Bailey - Test B8-R

Moment (kNm)	Rotation (Radians)
0.00	0.00000
3.60	0.00045
7.17	0.00090
11.00	0.00123
14.83	0.00157
18.65	0.00190
22.48	0.00213
26.31	0.00237
30.14	0.00260
34.61	0.00287
39.07	0.00313
43.53	0.00340
47.36	0.00385
51.19	0.00430
56.46	0.00480
61.71	0.00530
66.97	0.00575
72.24	0.00620
75.11	0.00680
77.98	0.00740
81.32	0.00815
84.68	0.00890
87.55	0.00945
90.42	0.01000
92.33	0.01105
94.25	0.01210
96.16	0.01290
98.08	0.01370
99.99	0.01475
101.90	0.01580
102.86	0.01665
103.82	0.01750
105.25	0.01840
106.69	0.01930
108.12	0.02015
109.56	0.02100
109.56	0.02200
109.56	0.02300
110.52	0.02375
111.47	0.02450
111.95	0.02540
112.43	0.02630
112.90	0.02725
113.38	0.02820

Table C29 : Moment-Rotation Data : Bailey - Test B9-L Connection Type : Extended end-plate connection to stiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test B9-Left

Beam size :	203 mm × 133 mm × 30 kg/m
Column size :	216 mm × 216 mm × 59 kg/m
End-plate :	422 × 165 × 19.0 mm

Major parameters :	Ct= -	Pt=4.500in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners :	7.9 mm column web stiffeners opposite both beam flanges
Beam fastening :	Weld of 11.1 mm for upper flange and 4.8 mm for remainder
Column fastening :	4 × 16.0 mm diameter HSFG bolts in tension and 4 × 19.0 mm diameter HSFG bolts in compression
Hole type :	Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:[B	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 80.5 kNm
Failure mode : Beam flange buckling

Remarks:
1) End-plate extended on tension side only
2) Two 16.0 mm diameter bolts used on either side of tension flange and four 19.0 mm bolts in compression
3) High shear loading
4) Only moment-deflection data available , moment-rotation data derived from references [4.23,4.24]

Table C29 : Moment-Rotation Data : Bailey - Test B9-L

Moment (kNm)	Rotation (Radians)
0.00	0.00000
3.83	0.00035
7.67	0.00070
11.50	0.00100
15.34	0.00130
18.70	0.00160
22.05	0.00190
24.45	0.00205
26.84	0.00220
29.50	0.00230
32.11	0.00240
34.99	0.00270
37.86	0.00300
40.50	0.00315
43.14	0.00330
46.02	0.00355
48.89	0.00380
51.28	0.00400
53.69	0.00420
55.60	0.00450
57.52	0.00480
60.39	0.00520
63.27	0.00560
65.19	0.00610
67.10	0.00660
68.78	0.00710
70.45	0.00760
71.65	0.00790
72.85	0.00820
74.06	0.00870
75.26	0.00920
76.45	0.00975
77.65	0.01030
78.61	0.01077
79.56	0.01123
80.52	0.01170

Table C30 : Moment-Rotation Data : Bailey - Test B9-R Connection Type : Extended end-plate connection to stiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test B9-Right

Beam size :	203 mm × 133 mm × 30 kg/m
Column size :	216 mm × 216 mm × 59 kg/m
End-plate :	422 × 165 × 19.0 mm

Major parameters :	Ct= -	Pt=4.500in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners :	7.9 mm column web stiffeners opposite both beam flanges
Beam fastening :	Weld of 11.1 mm for upper flange and 4.8 mm for remainder
Column fastening :	4 × 16.0 mm diameter HSFG bolts in tension and 4 × 19.0 mm diameter HSFG bolts in compression
Hole type :	Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 81.5 kNm
Failure mode : Beam flange buckling

Remarks:
1) End-plate extended on tension side only
2) Two 16.0 mm diameter bolts used on either side of tension flange and four 19.0 mm bolts in compression
3) High shear loading
4) Only moment-deflection data available , moment-rotation data derived from references [4.23,4.24]

Table C30 : Moment-Rotation Data : Bailey - Test B9-R

Moment (kNm)	Rotation (Radians)
0.00	0.00000
2.88	0.00030
5.75	0.00060
8.87	0.00085
11.99	0.00110
15.34	0.00140
18.70	0.00170
21.57	0.00195
24.45	0.00220
27.08	0.00245
29.71	0.00270
32.11	0.00290
34.52	0.00310
37.87	0.00340
41.22	0.00370
44.33	0.00395
47.45	0.00420
50.57	0.00450
53.69	0.00480
56.56	0.00530
59.44	0.00580
62.07	0.00655
64.70	0.00730
68.55	0.00830
69.74	0.00905
70.94	0.00980
72.14	0.01040
73.34	0.01100
74.77	0.01160
76.21	0.01220
77.17	0.01305
78.13	0.01390
79.09	0.01470
80.05	0.01550
80.28	0.01630
80.52	0.01710
81.01	0.01795
81.48	0.01880
81.72	0.01950
81.97	0.02020
81.97	0.02120

Table C31 : Moment-Rotation Data : Bailey - Test B10-L Connection Type : Extended end-plate connection to stiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test B10-Left

Beam size :	320 mm × 127 mm × 48 kg/m
Column size :	216 mm × 216 mm × 59 kg/m
End-plate :	422 × 177.5 × 20.6 mm

Major parameters :	Ct= -	Pt=4.500in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners :	7.9 mm column web stiffeners opposite both beam flanges
Beam fastening :	Weld of 16.0 mm for upper flange and 4.8 mm for remainder
Column fastening :	4 × 19.0 mm diameter HSFG bolts in tension and 4 × 25.4 mm diameter HSFG bolts in compression
Hole type :	Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 218.7 kNm
Failure mode : Beam flange buckling

Remarks:
1) End-plate extended on tension side only
2) Two 19.0 mm diameter bolts used on either side of tension flange and four 25.4 mm bolts in compression
3) High shear loading
4) Only moment-deflection data available , moment-rotation data derived from references [4.23,4.24]

Table C31 : Moment-Rotation Data : Bailey - Test B10-L

Moment (kNm)	Rotation (Radians)
0.00	0.00000
10.64	0.00020
21.25	0.00040
29.94	0.00055
38.64	0.00070
45.40	0.00080
52.16	0.00090
59.41	0.00110
66.65	0.00130
74.38	0.00135
82.10	0.00140
90.32	0.00160
98.53	0.00180
105.29	0.00205
112.06	0.00230
118.33	0.00240
124.61	0.00250
132.33	0.00330
140.06	0.00350
145.86	0.00400
151.65	0.00450
160.34	0.00520
164.69	0.00560
169.04	0.00600
172.91	0.00655
176.77	0.00710
181.60	0.00765
186.43	0.00820
189.00	0.00873
191.58	0.00927
194.16	0.00980
198.02	0.01050
201.89	0.01120
204.46	0.01177
207.04	0.01233
209.61	0.01290
212.02	0.01360
214.44	0.01430
216.86	0.01505
219.27	0.01580

Table C32 : Moment-Rotation Data : Bailey - Test B10-R Connection Type : Extended end-plate connection to stiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test B10-Right

Beam size :	320 mm × 127 mm × 48 kg/m
Column size :	216 mm × 216 mm × 59 kg/m
End-plate :	422 × 177.5 × 20.6 mm

Major parameters :	Ct= -	Pt=4.500in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners :	7.9 mm column web stiffeners opposite both beam flanges
Beam fastening :	Weld of 16.0 mm for upper flange and 4.8 mm for remainder
Column fastening :	4 × 19.0 mm diameter HSFG bolts in tension and 4 × 25.4 mm diameter HSFG bolts in compression
Hole type :	Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 205.0 kNm
Failure mode : Beam web buckling

Remarks:
1) End-plate extended on tension side only
2) Two 19.0 mm diameter bolts used on either side of tension flange and four 25.4 mm bolts in compression
3) High shear loading
4) Only moment-deflection data available , moment-rotation data derived from references [4.23,4.24]

Table C32 : Moment-Rotation Data : Bailey - Test B10-R

Moment (kNm)	Rotation (Radians)
0.00	0.00000
8.37	0.00020
16.74	0.00040
25.12	0.00060
34.29	0.00080
43.46	0.00100
51.19	0.00130
58.92	0.00160
69.55	0.00170
76.31	0.00195
83.07	0.00220
89.35	0.00235
95.63	0.00250
102.87	0.00275
110.12	0.00300
115.43	0.00315
120.74	0.00330
126.54	0.00355
132.33	0.00380
139.58	0.00435
146.82	0.00490
152.62	0.00550
158.41	0.00610
164.21	0.00680
170.00	0.00750
175.32	0.00835
180.63	0.00920
183.05	0.00985
185.46	0.01050
189.32	0.01125
193.19	0.01200
196.56	0.01295
199.95	0.01390
202.36	0.01490
204.78	0.01590
206.71	0.01680
208.64	0.01770
210.57	0.01870
212.51	0.01970
212.51	0.02045
212.51	0.02120

Table C33 : Moment-Rotation Data : Bailey - Test B11-L Connection Type : Extended end-plate connection to stiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test B11-Left

Beam size :	366 mm × 127 mm × 39 kg/m
Column size :	216 mm × 216 mm × 59 kg/m
End-plate :	422 × 165 × 22.2 mm

Major parameters :	Ct= -	Pt=4.250in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners :	7.9 mm column web stiffeners opposite both beam flanges
Beam fastening :	Weld of 11.1 mm for upper flange and 4.8 mm for remainder
Column fastening :	4 × 16.0 mm diameter HSFG bolts in tension and 4 × 19.0 mm diameter HSFG bolts in compression
Hole type :	Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 157.2 kNm
Failure mode : Beam web buckling

Remarks:
1) End-plate extended on tension side only
2) Two 16.0 mm diameter bolts used on either side of tension flange and four 19.0 mm bolts in compression
3) High shear loading
4) Only moment-deflection data available , moment-rotation data derived from references [4.23,4.24]

Table C33 : Moment-Rotation Data : Bailey - Test B11-L

Moment (kNm)	Rotation (Radians)
0.00	0.00000
6.96	0.00015
13.93	0.00030
20.65	0.00055
27.37	0.00080
33.61	0.00095
39.86	0.00110
46.10	0.00125
52.34	0.00140
58.60	0.00165
64.83	0.00190
69.63	0.00200
74.43	0.00210
80.19	0.00225
85.96	0.00240
92.20	0.00260
98.44	0.00280
102.77	0.00300
107.09	0.00320
111.41	0.00325
115.74	0.00330
120.54	0.00350
125.34	0.00370
132.06	0.00400
135.42	0.00420
138.77	0.00440
140.71	0.00470
142.63	0.00500
143.59	0.00527
144.55	0.00553
145.51	0.00580
148.39	0.00615
151.27	0.00650
153.19	0.00680
155.11	0.00710
156.07	0.00745
157.03	0.00780

Table C34 : Moment-Rotation Data : Bailey - Test B11-R
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test B11-Right

Beam size :	366 mm × 127 mm × 39 kg/m
Column size :	216 mm × 216 mm × 59 kg/m
End-plate :	422 × 165 × 22.2 mm

Major parameters :	Ct= -	Pt=4.250in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners :	7.9 mm column web stiffeners opposite both beam flanges
Beam fastening :	Weld of 11.1 mm for upper flange and 4.8 mm for remainder
Column fastening :	4 × 16.0 mm diameter HSFG bolts in tension and 4 × 19.0 mm diameter HSFG bolts in compression
Hole type :	Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 148.0 kNm
Failure mode : Beam web buckling

Remarks:
1) End-plate extended on tension side only
2) Two 16.0 mm diameter bolts used on either side of tension flange and four 19.0 mm bolts in compression
3) High shear loading
4) Only moment-deflection data available , moment-rotation data derived from references [4.23,4.24]

Table C34 : Moment-Rotation Data : Bailey - Test B11-R

Moment (kNm)	Rotation (Radians)
0.00	0.00000
7.21	0.00027
14.40	0.00053
21.61	0.00080
27.69	0.00100
33.77	0.00120
39.86	0.00140
46.58	0.00165
53.30	0.00190
58.59	0.00205
63.87	0.00220
69.16	0.00240
74.43	0.00260
79.23	0.00280
84.04	0.00300
91.24	0.00315
98.44	0.00330
104.69	0.00355
110.94	0.00380
114.78	0.00400
118.62	0.00420
122.46	0.00440
127.25	0.00455
132.06	0.00470
134.63	0.00493
137.18	0.00517
139.75	0.00540
142.14	0.00570
144.55	0.00600

Table C35 : Moment-Rotation Data : Bailey - Test B12-L Connection Type : Extended end-plate connection to stiffened column

Tested by : Bailey, J.R. [4.26]
Test identification: Test B12-Left

Beam size : 320 mm × 165 mm × 54 kg/m
Column size : 216 mm × 216 mm × 59 kg/m
End-plate : 422 × 190.5 × 27.0 mm

Major parameters :			
	Ct= -	Pt=4.375in	Pit=4.375in
	Cc=0.750in	Pc=2.500in	Pic=3.500in
	Lb= -	G=4.000in	Pi= -

Stiffeners : 7.9 mm column web stiffeners opposite both beam flanges
Beam fastening : Weld of 17.4 mm for upper flange and 4.8 mm for remainder
Column fastening : 4 × 22.2 mm diameter HSFG bolts in tension and 4 × 25.4 mm diameter HSFG bolts in compression
Hole type : Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
En[Bd-plate:	-	-

Average pretension force of bolts : -

Failure mode : Plasticity in beams and column flanges

Remarks:
1) End-plate extended on tension side only
2) Two 22.2 mm diameter bolts used on either side of tension flange and four 25.4 mm bolts in compression
3) High shear loading
4) Only moment-deflection data available , moment-rotation data derived from references [4.23,4.24]

Table C35 : Moment-Rotation Data : Bailey - Test B12-L

Moment (kNm)	Rotation (Radians)
0.00	0.00000
10.53	0.00037
21.05	0.00073
31.58	0.00110
44.97	0.00140
58.37	0.00170
69.85	0.00210
81.32	0.00250
94.72	0.00295
108.12	0.00340
120.56	0.00370
133.00	0.00400
144.48	0.00445
155.96	0.00490
170.32	0.00540
184.67	0.00590
195.19	0.00645
205.72	0.00700
215.29	0.00770
224.86	0.00840
240.16	0.00990
255.47	0.01150
265.04	0.01330
274.61	0.01560
280.35	0.01770
288.00	0.01950
293.74	0.02120
293.74	0.02330
299.48	0.02510
301.40	0.02740
303.31	0.02960
305.22	0.03110
306.18	0.03235
307.14	0.03360
307.14	0.03550
309.05	0.03730
309.05	0.03920
309.05	0.04120
312.88	0.04340
310.97	0.04520
314.79	0.04740
312.88	0.04850

Table C36 : Moment-Rotation Data : Bailey - Test B12-R Connection Type : Extended end-plate connection to stiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test B12-Right

Beam size :	320 mm × 165 mm × 54 kg/m
Column size :	216 mm × 216 mm × 59 kg/m
End-plate :	422 × 190.5 × 27.0 mm

Major parameters :	Ct= -	Pt=4.375in	Pit=4.375in
	Cc=0.750in	Pc=2.500in	Pic=3.500in
	Lb= -	G=4.000in	Pi= -

Stiffeners :	7.9 mm column web stiffeners opposite both beam flanges
Beam fastening :	Weld of 17.4 mm for upper flange and 4.8 mm for remainder
Column fastening :	4 × 22.2 mm diameter HSFG bolts in tension and 4 × 25.4 mm diameter HSFG bolts in compression
Hole type :	Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 297.6 kNm
Failure mode : Plasticity in beams and column flanges

Remarks:
1) End-plate extended on tension side only
2) Two 22.2 mm diameter bolts used on either side of tension flange and four 25.4 mm bolts in compression
3) High shear loading
4) Only moment-deflection data available , moment-rotation data derived from references [4.23,4.24]

Table C36 : Moment-Rotation Data : Bailey - Test B12-R

Moment (kNm)	Rotation (Radians)
0.00	0.00000
16.36	0.00035
32.72	0.00070
48.11	0.00105
63.51	0.00140
77.95	0.00165
92.37	0.00190
106.81	0.00220
121.25	0.00250
132.80	0.00275
144.34	0.00300
154.93	0.00340
165.51	0.00380
179.95	0.00405
194.38	0.00430
205.93	0.00480
217.48	0.00530
226.13	0.00590
234.80	0.00650
246.35	0.00735
257.89	0.00820
266.55	0.00900
275.21	0.00980
290.61	0.01190
300.22	0.01370
307.93	0.01590
315.62	0.01790
321.40	0.02010
322.36	0.02130
323.32	0.02250
329.10	0.02470
329.10	0.02680
332.94	0.02860
332.94	0.03040
332.94	0.03240
336.80	0.03420
336.80	0.03650
338.72	0.03830
338.72	0.04020
336.80	0.04210
340.65	0.04420
340.65	0.04590
340.65	0.04720

Table C37 : Moment-Rotation Data : Bailey - Test B13-L Connection Type : Extended end-plate connection to stiffened column

Tested by : Bailey, J.R. [4.26]
Test identification: Test B13-Left

Beam size : 366 mm × 171 mm × 67 kg/m
Column size : 216 mm × 216 mm × 59 kg/m
End-plate : 422 × 190.5 × 31.8 mm

Major parameters :	Ct= -	Pt=4.625in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners : 7.9 mm column web stiffeners opposite both beam flanges
Beam fastening : Weld of 17.4 mm for upper flange and 4.8 mm for remainder
Column fastening : 4 × 25.4 mm diameter HSFG bolts in tension and 4 × 28.6 mm diameter HSFG bolts in compression
Hole type : Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : Not completed

Remarks:
1) End-plate extended on tension side only
2) Two 25.4 mm diameter bolts used on either side of tension flange and four 28.6 mm bolts in compression
3) High shear loading
4) Only moment-deflection data available , moment-rotation data derived from references [4.23,4.24]

Table C37 : Moment-Rotation Data : Bailey - Test B13-L

Moment (kNm)	Rotation (Radians)
0.00	0.00000
4.78	0.00010
9.55	0.00020
14.33	0.00030
19.10	0.00040
23.87	0.00050
28.65	0.00060
33.01	0.00070
37.38	0.00080
46.12	0.00100
59.21	0.00130
72.58	0.00170
85.96	0.00210
101.23	0.00250
116.51	0.00190
131.79	0.00330
147.07	0.00370
160.44	0.00400
173.81	0.00430
186.23	0.00455
198.64	0.00480
213.92	0.00530
227.30	0.00580
240.66	0.00660
249.26	0.00700
257.86	0.00740
273.13	0.00820
284.60	0.00900
291.28	0.00960
297.97	0.01020
302.74	0.01070
307.51	0.01120
311.33	0.01170
315.15	0.01220
318.97	0.01280
322.80	0.01340
325.66	0.01395
328.53	0.01450
329.48	0.01510
330.44	0.01570
331.39	0.01625
332.35	0.01680

Table C38 : Moment-Rotation Data : Bailey - Test B13-R Connection Type : Extended end-plate connection to stiffened column

Tested by :	Bailey, J.R. [4.26]
Test identification:	Test B13-Right

Beam size :	366 mm × 171 mm × 67 kg/m
Column size :	216 mm × 216 mm × 59 kg/m
End-plate :	422 × 190.5 × 31.8 mm

Major parameters :	Ct= -	Pt=4.625in	Pit= -
	Cc= -	Pc= -	Pic= -
	Lb= -	G=4.000in	Pi= -

Stiffeners :	7.9 mm column web stiffeners opposite both beam flanges
Beam fastening :	Weld of 17.4 mm for upper flange and 4.8 mm for remainder
Column fastening :	4 × 25.4 mm diameter HSFG bolts in tension and 4 × 28.6 mm diameter HSFG bolts in compression
Hole type :	Not drilled

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : Not completed

Remarks:
1) End-plate extended on tension side only
2) Two 25.4 mm diameter bolts used on either side of tension flange and four 28.6 mm bolts in compression
3) High shear loading
4) Only moment-deflection data available , moment-rotation data derived from references [4.23,4.24]

Table C38 : Moment-Rotation Data : Bailey - Test B13-R

Moment (kNm)	Rotation (Radians)
0.00	0.00000
14.72	0.00020
29.44	0.00040
43.68	0.00080
57.93	0.00120
72.17	0.00150
86.42	0.00180
100.67	0.00205
114.90	0.00230
126.30	0.00260
137.70	0.00290
149.09	0.00315
160.49	0.00340
175.69	0.00370
184.23	0.00395
192.78	0.00420
202.27	0.00480
215.57	0.00540
226.97	0.00590
242.15	0.00690
251.65	0.00760
261.15	0.00830
267.80	0.00895
274.44	0.00960
281.10	0.01015
287.74	0.01070
293.44	0.01145
299.14	0.01220
302.94	0.01305
306.73	0.01390
310.53	0.01465
314.33	0.01540
316.24	0.01620
318.13	0.01700
319.08	0.01785
320.03	0.01870
320.03	0.01950
320.03	0.02030

Table C39 : Moment-Rotation Data : Zoetemeijer - Test 9
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Zoetemeijer,P. [4.12]
Test identification:	Test 9

Beam size :	IPE 300
Column size :	HE 200A
End-plate :	400 × 200 × 32 mm

Major parameters :	Ct=30mm	Pt=90mm	Pit=0
	Cc+Pc=80mm	Pc= -	Pic=0
	Lb=290mm	G=100mm	Pi= 200mm

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	6 × M20 Grade 10.9 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	264.5	451.0
Beam web:	-	-
Column flange:	301.0	460.5
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : Not preloaded

Failure moment: -
Failure mode : -

Remarks: 1) End-plate extended on tension side only
2) Moment-rotation curve not provided

Table C40 : Moment-Rotation Data : Zoetemeijer - Test 10
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Zoetemeijer,P. [4.12]
Test identification:	Test 10

Beam size :	IPE 300
Column size :	HE 200A
End-plate :	400 × 200 × 32 mm

Major parameters :	Ct= 30mm	Pt=90mm	Pit=0
	Cc+Pc=80mm	Pc= -	Pic=0
	Lb=290mm	G=100mm	Pi=200mm

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	6 × M20 Grade 10.9 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	264.5	451.0
Beam web:	-	-
Column flange:	301.0	460.5
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : Not preloaded

Failure moment: -
Failure mode : -

Remarks:
1) End-plate extended on tension side only
2) 300 × 80 × 13 mm flange backing plates
3) Moment-rotation curve not provided

Table C41 : Moment-Rotation Data : Zoetemeijer - Test 20 Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Zoetemeijer,P. [4.12]
Test identification:	Test 20

Beam size :	IPE 400
Column size :	HE 300A
End-plate :	510 × 270 × 32 mm

Major parameters :	Ct=-	Pt=-	Pit=-
	Cc=-	Pc=-	Pic=-
	Lb=-	G=-	Pi=-

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	6 × M20 Grade 10.9 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	317.0	417.0
Beam web:	-	-
Column flange:	257.0	399.0
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : Not preloaded

Failure moment: -
Failure mode : -

Remarks: 1) End-plate extended on tension side only
2) Moment-rotation curve not provided

Table C42 : Moment-Rotation Data : Zoetemeijer - Test 21
Connection Type : Extended end-plate connection to unstiffened column

Tested by : Zoetemeijer,P. [4.12]
Test identification: Test 21

Beam size : IPE 400
Column size : HE 300A
End-plate : 510 × 270 × 32 mm

Major parameters :	Ct=-	Pt=-	Pit=-
	Cc=-	Pc=-	Pic=-
	Lb=-	G=-	Pi=-

Stiffeners : No
Beam fastening : Weld
Column fastening : 6 × M20 Grade 10.9 bolts
Hole type : -

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	317.0	417.0
Beam web:	-	-
Column flange:	257.0	399.0
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : Not preloaded

Failure moment: -
Failure mode : Web buckling observed after testing

Remarks: 1) End-plate extended on tension side only
2) 350 × 110 × 16 mm flange backing plates
3) Moment-rotation curve not provided

Table C43 : Moment-Rotation Data : Zoetemeijer - Test M3A Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Zoetemeijer,P. [4.12]
Test identification:	Test M3A

Beam size :	IPE 300
Column size :	HE 240A
End-plate :	400 × 180 × 22 mm

Major parameters :	Ct=-	Pt=-	Pit=-
	Cc=-	Pc=-	Pic=-
	Lb=-	G=-	Pi=-

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	6 × M20 Grade 10.9 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	264.5	451.0
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : Not preloaded

Failure moment: -
Failure mode : Crack in weld

Remarks: 1) End-plate extended on tension side only
2) Connection tested in portal frame
3) Moment-rotation curve provided in reference [4.12]

Table C43 : Moment-Rotation Data : Zoetemeijer - Test M3A

Moment (kNm)	Rotation (Radians)
0.00	0.00000
52.00	0.00115
68.00	0.00200
80.00	0.00225
96.50	0.00325
106.00	0.00495
120.00	0.00795
135.00	0.01190
146.50	0.02000
148.00	0.02180
144.50	0.02480
144.50	0.03220
144.00	0.04000
141.00	0.04660
138.00	0.05305

Table C44 : Moment-Rotation Data : Zoetemeijer - Test M3B Connection Type : Extended end-plate connection to unstiffened column

Tested by : Zoetemeijer,P. [4.12]
Test identification: Test M3B

Beam size : IPE 300
Column size : HE 240A
End-plate : 430 × 180 × 22 mm

Major parameters :	Ct=-	Pt=-	Pit=-
	Cc=-	Pc=-	Pic=-
	Lb=-	G=-	Pi=-

Stiffeners : No
Beam fastening : Weld
Column fastening : 6 × M20 Grade 10.9 bolts
Hole type : -

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	264.5	451.0
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : Not preloaded

Failure moment: -
Failure mode : -

Remarks: 1) End-plate extended on tension side only
2) Connection tested in portal frame
3) Moment-rotation curve provided in reference [4.12]

Table C44 : Moment-Rotation Data : Zoetemeijer - Test M3B

Moment (kNm)	Rotation (Radians)
0.00	0.00000
52.00	0.00115
68.00	0.00200
80.00	0.00300
94.00	0.00420
108.00	0.00600
118.50	0.00845
125.00	0.01290
132.00	0.01685
130.00	0.02000
136.00	0.02505
132.00	0.02975
127.50	0.03570
140.00	0.04000
97.50	0.04860
92.00	0.05355
82.50	0.06000
80.00	0.06400

Table C45 : Moment-Rotation Data : Zoetemeijer - Test M3C Connection Type : Extended end-plate connection to unstiffened column

Tested by : Zoetemeijer,P. [4.12]
Test identification: Test M3C

Beam size : IPE 300
Column size : HE 240A
End-plate : 430 × 180 × 22 mm

Major parameters :	Ct=-	Pt=-	Pit=-
	Cc=-	Pc=-	Pic=-
	Lb=-	G=-	Pi=-

Stiffeners : No
Beam fastening : Weld
Column fastening : 6 × M20 Grade 10.9 bolts
Hole type : -

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	261.5	451.0
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : -

Remarks: 1) End-plate extended on tension side only
2) Connection tested in portal frame
3) Moment-rotation curve not provided

Table C46 : Moment-Rotation Data : Zoetemeijer - Test M3D Connection Type : Extended end-plate connection to unstiffened column

Tested by : Zoetemeijer,P. [4.12]
Test identification: Test M3D

Beam size : IPE 300
Column size : HE 240A
End-plate : 430 × 180 × 22 mm

Major parameters :	Ct=-	Pt=-	Pit=-
	Cc=-	Pc=-	Pic=-
	Lb=-	G=-	Pi=-

Stiffeners : No
Beam fastening : Weld
Column fastening : 6 × M20 Grade 10.9 bolts
Hole type : -

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	261.5	451.0
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : Not preloaded

Failure moment: 160 kNm
Failure mode : Crack in weld

Remarks: 1) End-plate extended on tension side only
2) Connection tested in portal frame
3) Moment-rotation curve provided in reference [4.12]

Table C46 : Moment-Rotation Data : Zoetemeijer - Test M3D

Moment (kNm)	Rotation (Radians)
0.00	0.00000
44.50	0.00075
80.00	0.00125
106.00	0.00400
120.00	0.00645
128.00	0.00940
136.00	0.01365
148.00	0.02000
154.00	0.02330
160.00	0.02880
162.50	0.03025
128.00	0.03520
100.00	0.04000
80.00	0.04365
48.50	0.04960
35.00	0.06000
26.50	0.06700

Table C47 : Moment-Rotation Data : Packer - Test J1
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Packer, J.A. [4.3]
Test identification:	Test J1

Beam size :	245 × 102 × UB 22
Column size :	152 × 152 × UC 37
End-plate :	368 × 150 × 15 mm

Major parameters :	Ct=40mm	Pt=97mm	Pit=0
	Cc=31.5mm	Pc=48.5mm	Pic=0
	Lb=248mm	G=86mm	Pi=151mm

Stiffeners :	8 mm compression stiffeners
Beam fastening :	Fillet welded all round
Column fastening :	6 × M16 HSFG bolts
Hole type :	20.6 mm oversize tension side holes 17.5 mm oversize compression side holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	330.1	496.7
Beam web:	309.2	486.6
Column flange:	281.9	487.0
Column web:	307.5	495.0
End-plate:	239.2	423.8

Average pretension force of bolts : 45 kN

Failure moment: 97.3 kNm
Failure mode : Local beam flange buckling

Remarks:
1) End-plate extended on tension side only
2) Two M16 diameter bolts used on either side of tension flange and two M16 bolts in compression
3) Washers under bolt heads and nuts
4) Moment-rotation curve provided in references[4.3,4.4]

Table C47 : Moment-Rotation Data : Packer - Test J1

Moment (kNm)	Rotation (Radians)
0.00	0.00000
7.40	0.00100
15.70	0.00155
20.00	0.00210
23.10	0.00250
26.05	0.00335
39.25	0.00425
40.00	0.00450
47.15	0.00605
50.10	0.00745
54.00	0.00895
56.00	0.01000
57.95	0.01100
60.00	0.01215
61.90	0.01390
65.80	0.01640
68.75	0.01940
70.70	0.02000
73.70	0.02355
78.60	0.03000
80.00	0.03170
83.75	0.04000
87.45	0.04810
88.50	0.05000
92.35	0.05700
96.25	0.06000
97.20	0.07000
97.30	0.07655
97.50	0.08000
97.65	0.09000
95.00	0.10000
90.15	0.10500

Table C48 : Moment-Rotation Data : Packer - Test J2 Connection Type : Extended end-plate connection to unstiffened column

Tested by : Packer, J.A. [4.3]
Test identification: Test J2

Beam size : 245 × 102 × UB 22
Column size : 152 × 152 × UC 23
End-plate : 368 × 150 × 15 mm

Major parameters :			
	Ct=40mm	Pt=97mm	Pit=0
	Cc=31.5mm	Pc=48.5mm	Pic=0
	Lb=248mm	G=86mm	Pi=151mm

Stiffeners : 8 mm compression column web stiffeners
Beam fastening : Fillet welded all round
Column fastening : 6 × M16 HSFG bolts
Hole type : 20.6 mm oversize tension side holes
17.5 mm oversize compression side holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	330.1	496.7
Beam web:	309.2	486.6
Column flange:	334.5	499.0
Column web:	363.4	535.3
End-plate:	239.2	423.8

Average pretension force of bolts : 35 kN

Failure moment: 79.0 kNm
Failure mode : Local beam flange buckling

Remarks:
1) End-plate extended on tension side only
2) Two M16 diameter bolts used on either side of tension flange and two M16 bolts in compression
3) Washers under nuts only
4) Moment-rotation curve provided in references [4.3,4.4]

Table C48 : Moment-Rotation Data : Packer - Test J2

Moment (kNm)	Rotation (Radians)
0.00	0.00000
3.95	0.00075
7.40	0.00145
10.90	0.00245
14.80	0.00270
18.75	0.00345
20.00	0.00400
21.00	0.00410
22.70	0.00450
25.20	0.00510
26.80	0.00570
30.15	0.00670
34.05	0.00845
36.65	0.01000
38.05	0.01090
40.00	0.01190
41.50	0.01340
49.40	0.01935
50.35	0.02000
53.80	0.02355
60.00	0.03000
61.35	0.03295
64.70	0.04000
65.35	0.04330
66.95	0.05000
69.15	0.06000
70.10	0.06520
70.65	0.07000
72.10	0.08000
73.10	0.08380
74.05	0.09000
75.55	0.10000
77.00	0.11000
78.50	0.12000
79.00	0.12620
78.05	0.13000

Table C49 : Moment-Rotation Data : Packer - Test J3 Connection Type : Extended end-plate connection to stiffened column

Tested by :	Packer, J.A. [4.3]
Test identification:	Test J3

Beam size :	245 × 102 × UB 22
Column size :	152 × 152 × UC 23
End-plate :	368 × 150 × 15 mm

Major parameters :	Ct=40mm	Pt=97mm	Pit=0
	Cc=31.5mm	Pc=48.5mm	Pic=0
	Lb=248mm	G=86mm	Pi=151mm

Stiffeners :	8 mm compression stiffeners with fillet all rond and 6.3 mm tension stiffeners welded only to the column flange
Beam fastening :	Fillet welded all round
Column fastening :	6 × M16 HSFG bolts
Hole type :	20.6 mm oversize tension side holes 17.5 mm oversize compression side holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	330.1	496.7
Beam web:	309.2	486.6
Column flange:	334.5	499.0
Column web:	363.4	535.3
End-plate:	239.2	423.8

Average pretension force of bolts : 35 kN

Failure moment: 96.5 kNm
Failure mode : Local beam flange buckling

Remarks:
1) End-plate extended on tension side only
2) Two M16 diameter bolts used on either side of tension flange and two M16 bolts in compression
3) Washers under bolt head and nuts
4) Moment-rotation curve provided in references [4.3,4.4]

Table C49 : Moment-Rotation Data : Packer - Test J3

Moment (kNm)	Rotation (Radians)
0.00	0.00000
7.40	0.00125
14.80	0.00200
20.00	0.00295
22.70	0.00325
30.15	0.00435
33.85	0.00570
37.50	0.00745
40.00	0.00890
41.50	0.01000
42.60	0.01050
46.40	0.01315
52.80	0.02000
60.00	0.02830
61.25	0.03000
68.15	0.04000
69.40	0.04170
74.15	0.05000
76.05	0.05305
80.00	0.05880
80.60	0.06000
83.95	0.06720
85.95	0.07000
89.90	0.08000
91.85	0.08690
92.35	0.09000
94.00	0.10000
96.05	0.11000
96.50	0.12000
93.60	0.12840

Table C50 : Moment-Rotation Data : Packer - Test J4
Connection Type : Extended end-plate connection to unstiffened column

Tested by : Packer, J.A. [4.3]
Test identification: Test J4

Beam size : 245 × 102 × UB 22
Column size : 152 × 152 × UC 23
End-plate : 368 × 150 × 15 mm

Major parameters :	Ct=40mm	Pt=97mm	Pit=0
	Cc=31.5mm	Pc=48.5mm	Pic=0
	Lb=248mm	G=86mm	Pi=151mm

Stiffeners : No
Beam fastening : Fillet welded all round
Column fastening : 6 × M16 HSFG bolts
Hole type : 20.6 mm oversize tension side holes
17.5 mm oversize compression side holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	330.1	496.7
Beam web:	309.2	486.6
Column flange:	334.5	499.0
Column web:	363.4	535.3
End-plate:	239.2	423.8

Average pretension force of bolts : 35 kN

Failure moment: 82.6 kNm
Failure mode : Bolt faillure

Remarks: 1) End-plate extended on tension side only
2) Two M16 diameter bolts used on either side of tension flange and two M16 bolts in compression
3) Washers under bolt head and nuts
4) Axial load in column of 35
5)Moment-rotation data provided in refernce[4.24]

Table C50 : Moment-Rotation Data : Packer - Test J4

Moment (kNm)	Rotation (Radians)
0.00	0.00000
10.80	0.00170
20.00	0.00320
22.60	0.00415
26.50	0.00570
31.00	0.00865
32.90	0.01000
34.40	0.01200
40.00	0.01850
41.30	0.02000
44.20	0.02450
44.70	0.03000
49.15	0.03160
53.60	0.03900
54.10	0.04000
58.00	0.04540
60.00	0.04790
61.50	0.05000
64.90	0.05630
66.85	0.06000
70.35	0.07000
70.85	0.07060
73.75	0.08000
74.75	0.08225
76.50	0.09000
77.70	0.09380
80.00	0.09730
80.35	0.10000
81.15	0.10220
82.15	0.11000
82.60	0.11015
80.75	0.11360
74.75	0.11655

Table C51 : Moment-Rotation Data : Packer - Test J5
Connection Type : Extended end-plate connection to unstiffened column

Tested by : Packer, J.A. [4.3]
Test identification: Test J5

Beam size : 245 × 102 × UB 22
Column size : 152 × 152 × UC 30
End-plate : 368 × 150 × 15 mm

Major parameters :	Ct=40mm	Pt=97mm	Pit=0
	Cc=31.5mm	Pc=48.5mm	Pic=0
	Lb=248mm	G=86mm	Pi=151mm

Stiffeners : No
Beam fastening : Fillet welded all round
Column fastening : 6 × M16 HSFG bolts
Hole type : 20.6 mm oversize tension side holes
17.5 mm oversize compression side holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	330.1	496.7
Beam web:	309.2	486.6
Column flange:	353.8	554.6
Column web:	374.3	564.3
End-plate:	239.2	423.8

Average pretension force of bolts : 33 kN

Failure moment: 87.8 kNm
Failure mode : Local beam flange buckling

Remarks:
1) End-plate extended on tension side only
2) Two M16 diameter bolts used on either side of tension flange and two M16 bolts in compression
3) Washers under bolt head and nuts
4) Moment-rotation curve provided in reference [4.3,4.4]

Table C51 : Moment-Rotation Data : Packer - Test J5

Moment (kNm)	Rotation (Radians)
0.00	0.00000
6.90	0.00095
14.80	0.00160
20.00	0.00210
22.30	0.00250
30.65	0.00345
34.60	0.00450
38.50	0.05445
40.00	0.00645
42.50	0.00795
45.40	0.01000
49.90	0.01290
53.35	0.01585
58.30	0.02000
60.00	0.02225
62.00	0.02375
66.70	0.03000
69.15	0.03515
71.15	0.04000
75.05	0.05000
76.05	0.05350
77.55	0.06000
80.00	0.06760
81.00	0.07000
84.00	0.08000
86.90	0.09000
87.40	0.10000
87.85	0.10395
86.90	0.11000
84.95	0.11185

Table C52 : Moment-Rotation Data : Ioannides - Test 1
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Ioannides, S.A. [4.27]
Test identification:	Test 1

Beam size :	W14 × 22
Column size :	W8 × 35
End-plate :	20.50 × 8.00 × 0.625 in.

Major parameters :	Ct=1.75in	Pt=3.50in	Lb=17.25in
	Cc=1.75in	Pc=3.50in	Li=10.25in
	G=5.50in		

Stiffeners :	No
Beam fastening :	Weld with a minimum of 1 in. return at the ends
Column fastening :	8 × 3/4 in. A325 bolts
Hole type :	13/16 in. oversize drilled holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 84.0 kips.ft
Failure mode : Column tension flange yield

Remarks: 1) End-plate extended on tension and compression sides
2) Py/3 axial load on column
4) Moment-rotation data provided in reference [4.23,4.24]

Table C52 : Moment-Rotation Data : Ioannides - Test 1

Moment (kips.ft)	Rotation (Radians)
0.000	0.00000
3.475	0.00019
6.983	0.00038
10.417	0.00057
13.767	0.00079
17.117	0.00100
20.467	0.00121
23.250	0.00136
26.033	0.00151
28.825	0.00165
32.150	0.00184
35.483	0.00202
38.758	0.00240
41.483	0.00272
44.208	0.00304
47.708	0.00363
51.133	0.00476
53.808	0.00585
56.483	0.00698
58.250	0.00786
60.017	0.00873
61.542	0.00971
63.067	0.01069
64.900	0.01194
66.733	0.01319
68.367	0.01444
69.992	0.01569
71.625	0.01693
72.758	0.01794
73.892	0.01894
75.025	0.01995
76.158	0.02095
77.108	0.02202
78.058	0.02310
79.017	0.02417
79.967	0.02524

Table C53 : Moment-Rotation Data : Ioannides - Test 2 Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Ioannides, S.A. [4.27]
Test identification:	Test 2

Beam size :	W18 × 35
Column size :	W10 × 49
End-plate :	24.75 × 8.00 × 0.625 in.

Major parameters :	Ct=1.625in	Pt=3.750in	Lb=21.500in
	Cc=1.625in	Pc=3.750in	Li=14.000in
	G=5.50in		

Stiffeners :	No
Beam fastening :	Weld with a minimum of 1 in. return at the ends
Column fastening :	8 × 7/8 in. A325 bolts
Hole type :	15/16 in. oversize drilled holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment:	174.0 kips.ft
Failure mode :	Column tension flange yield

Remarks:
1) End-plate extended on tension and compression sides
2) Py/3 axial load on column
3) Moment-rotation data provided in references [4.23,4.24]

Table C53 : Moment-Rotation Data : Ioannides - Test 2

Moment (kips.ft)	Rotation (Radians)
0.000	0.00000
9.317	0.00002
18.625	0.00003
27.942	0.00005
37.250	0.00007
46.567	0.00008
54.758	0.00024
62.950	0.00039
70.175	0.00052
77.400	0.00065
84.625	0.00087
92.567	0.00113
100.250	0.00152
107.933	0.00198
116.808	0.00262
125.175	0.00349
131.850	0.00443
135.650	0.00509
139.458	0.00575
142.767	0.00650
146.083	0.00724
149.500	0.00818
152.917	0.00911
156.083	0.01008
159.258	0.01105
162.200	0.01186
165.150	0.01268
167.725	0.01354
170.308	0.01441
172.883	0.01527
175.550	0.01604
178.217	0.01682
180.883	0.01759
183.142	0.01841
185.408	0.01923
187.667	0.02004
190.592	0.02112
193.508	0.02219

Table C54 : Moment-Rotation Data : Ioannides - Test 3
Connection Type : Extended end-plate connection to unstiffened column

Tested by : Ioannides, S.A. [4.27]
Test identification: Test 3

Beam size : W24 × 55
Column size : W14 × 48
End-plate : 19.25 × 8.00 × 0.875 in.

Major parameters :	Ct=1.500in	Pt=4.000in	Lb=27.250in
	Cc=1.500in	Pc=4.000in	Li=19.250in
	G=5.50in		

Stiffeners : No
Beam fastening : Weld with a minimum of 1 in. return at the ends
Column fastening : 8 × 1.0 in. A325 bolts
Hole type : 1.0625 in. oversize drilled holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 224.0 kips.ft
Failure mode : Column tension flange yield

Remarks: 1) End-plate extended on tension and compression sides
2) Py/3 axial load on column
3) Moment-rotation data provided in references [4.23,4.24]

Table C54 : Moment-Rotation Data : Ioannides - Test 3

Moment (kips.ft)	Rotation (Radians)
0.000	0.00000
8.158	0.00002
16.308	0.00005
24.467	0.00007
31.600	0.00009
38.733	0.00012
46.042	0.00015
53.342	0.00019
60.308	0.00025
67.275	0.00031
74.400	0.00041
81.533	0.00051
89.350	0.00064
97.158	0.00078
106.158	0.00100
115.150	0.00122
124.658	0.00146
134.167	0.00170
142.550	0.00206
149.433	0.00241
155.200	0.00269
160.967	0.00297
167.242	0.00335
173.517	0.00374
179.783	0.00418
186.058	0.00463
191.142	0.00509
196.225	0.00555
200.958	0.00610
205.700	0.00666
209.425	0.00715
213.142	0.00764
215.675	0.00820
218.200	0.00876
220.225	0.00934
222.242	0.00992
223.925	0.01044
225.608	0.01095
226.608	0.01136
227.617	0.01177

Table C55 : Moment-Rotation Data : Ioannides - Test 4
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Ioannides, S.A. [4.27]
Test identification:	Test 4

Beam size :	W14 × 22
Column size :	W8 × 35
End-plate :	20.50 × 8.00 × 0.875 in.

Major parameters :	Ct=1.75in	Pt=3.50in	Lb=17.25in
	Cc=1.75in	Pc=3.50in	Li=10.25in
	G=5.50in		

Stiffeners :	No
Beam fastening :	Weld with a minimum of 1 in. return at the ends
Column fastening :	8 × 3/4 in. A325 bolts
Hole type :	13/16 in. oversize drilled holes

Measured Material Properties:
All sections steel grade ASTM A36

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment:	112.0 kips.ft
Failure mode :	Column tension flange yield

Remarks:
1) End-plate extended on tension and compression sides
2) Py/3 axial load on column
3) Moment-rotation data provided in reference [4.23,4.24]

Table C55 : Moment-Rotation Data : Ioannides - Test 4

Moment (kips.ft)	Rotation (Radians)
0.000	0.00000
4.492	0.00013
9.475	0.00017
14.450	0.00021
19.425	0.00026
22.458	0.00027
25.492	0.00027
30.717	0.00043
35.700	0.00120
44.292	0.00155
47.517	0.00190
50.750	0.00250
53.992	0.00309
57.358	0.00413
60.733	0.00518
64.675	0.00674
68.608	0.00829
72.058	0.00990
75.517	0.01152
78.550	0.01330
81.583	0.01509
84.558	0.01696
87.533	0.01883
90.150	0.02070
92.758	0.02257
94.850	0.02463
96.933	0.02669
99.025	0.02875
101.117	0.03081
103.200	0.03287
105.292	0.03492
107.375	0.03698
109.467	0.03904
110.550	0.04052
111.633	0.04200

Table C56 : Moment-Rotation Data : Ioannides - Test 5
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Ioannides, S.A. [4.27]
Test identification:	Test 5

Beam size :	W18 × 35
Column size :	W10 × 49
End-plate :	24.75 × 8.00 × 1.250 in.

Major parameters :	Ct=1.625in	Pt=3.750in	Lb=21.500in
	Cc=1.625in	Pc=3.750in	Li=14.000in
	G=5.500in		

Stiffeners :	No
Beam fastening :	Weld with a minimum of 1 in. return at the ends
Column fastening :	8 × 7/8 in. A325 bolts
Hole type :	15/16 in. oversize drilled holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment:	216.0 kips.ft
Failure mode :	Column tension flange yield

Remarks: 1) End-plate extended on tension and compression sides
2) Py/3 axial load on column
3) Moment-rotation data provided in references[4.23,4.24]

Table C56 : Moment-Rotation Data : Ioannides - Test 5

Moment (kips.ft)	Rotation (Radians)
0.000	0.00000
8.158	0.00014
16.317	0.00029
24.475	0.00043
33.200	0.00066
41.925	0.00089
51.250	0.00124
60.575	0.00159
68.450	0.00199
76.317	0.00239
84.667	0.00288
93.025	0.00337
100.883	0.00402
108.750	0.00466
116.367	0.00550
123.983	0.00633
130.500	0.00722
137.025	0.00810
142.450	0.00930
147.875	0.01050
153.775	0.01190
159.675	0.01330
164.850	0.01481
170.025	0.01632
173.867	0.01758
177.708	0.01885
182.517	0.02046
187.325	0.02207
190.917	0.02377
194.508	0.02547
198.217	0.02732
201.925	0.02917
204.392	0.03055
206.858	0.03192
209.325	0.03329
212.067	0.03501
214.808	0.03672
216.475	0.03786
218.142	0.03901

Table C57 : Moment-Rotation Data : Ioannides - Test 6
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Ioannides, S.A. [4.27]
Test identification:	Test 6

Beam size :	W24 × 55
Column size :	W14 × 48
End-plate :	30.50 × 8.00 × 1.250 in.

Major parameters :	Ct=1.50in	Pt=4.00in	Lb=27.25in
	Cc=1.50in	Pc=4.00in	Li=19.25in
	G=5.50in		

Stiffeners :	No
Beam fastening :	Weld with a minimum of 1 in. return at the ends
Column fastening :	8 × 1.0 in. A325 bolts
Hole type :	1.0625 in. oversize drilled holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 272.0 kips.ft
Failure mode : Column web buckling

Remarks:
1) End-plate extended on tension and compression sides
2) Py/3 axial load on column
3) Moment-rotation data provided in references [4.23,4.24]

Table C57 : Moment-Rotation Data : Ioannides - Test 6

Moment (kips.ft)	Rotation (Radians)
0.000	0.00000
127.934	0.00010
133.767	0.00011
137.825	0.00017
142.542	0.00031
146.917	0.00045
151.958	0.00063
157.642	0.00090
164.667	0.00123
171.333	0.00164
177.675	0.00198
182.150	0.00230
186.625	0.00262
194.575	0.00322
199.033	0.00360
203.492	0.00397
208.608	0.00441
213.725	0.00484
217.833	0.00524
221.942	0.00564
226.533	0.00611
231.125	0.00658
234.950	0.00699
238.767	0.00740
242.592	0.00781
246.483	0.00833
250.375	0.00885
254.092	0.00939
257.808	0.00993
261.025	0.01043
264.250	0.01092
267.950	0.01150
271.650	0.01208

Table C58 : Moment-Rotation Data : Dews - Test 1
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Dews, R.J. [4.28]
Test identification:	Test 1

Beam size :	W14 × 22
Column size :	W8 × 31
End-plate :	20.429 × 8.000 × 1.000 in.

Major parameters :	Ct=1.8075in	Pt=3.500in	Lb=16.814in
	Cc=1.8075in	Pc=3.500in	Li=9.8140in
	G=5.500in		

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	8 × 3/4 in. A325 bolts
Hole type :	15/16 in. oversize holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	48.10	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : -

Remarks: 1) End-plate extended on tension and compression sides
2) Moment-rotation data provided in references [4.23,4.24]

Table C58 : Moment-Rotation Data : Dews - Test 1

Moment (kips.ft)	Rotation (Radians)
0.000	0.00000
3.183	0.00071
6.367	0.00143
9.208	0.00210
12.058	0.00278
16.408	0.00370
20.767	0.00462
24.842	0.00554
28.917	0.00647
32.425	0.00717
35.942	0.00788
39.742	0.00876
43.533	0.00965
50.458	0.01116
52.925	0.01222
55.192	0.01365
58.200	0.01646
62.158	0.01963
65.708	0.02300
67.667	0.02523
69.633	0.02747
71.725	0.03040
73.825	0.03333
75.742	0.03608
77.667	0.03883
79.317	0.04163
80.967	0.04444
82.350	0.04707
83.733	0.04971
85.117	0.05235
86.392	0.05499
87.667	0.05763
88.933	0.06028
90.108	0.06317
91.275	0.06606
92.450	0.06895
93.467	0.07157
94.475	0.07419
95.492	0.07680

Table C59 : Moment-Rotation Data : Dews - Test 2
Connection Type : Extended end-plate connection to unstiffened column

Tested by : Dews, R.J. [4.28]
Test identification: Test 2

Beam size : W14 × 22
Column size : W8 × 31
End-plate : 20.531 × 8.000 × 1.000 in.

Major parameters :	Ct=1.8075in	Pt=3.500in	Lb=16.916in
	Cc=1.8075in	Pc=3.500in	Li=9.916in
	G=5.531in		

Stiffeners : No
Beam fastening : Weld
Column fastening : 8 × 3/4 in. A325 bolts
Hole type : 15/16 in. oversize holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	36.90	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : -

Remarks: 1) End-plate extended on tension and compression sides
2) Moment-rotation data provided in references [4.23,4.24]

Table C59 : Moment-Rotation Data : Dews - Test 2

Moment (kips.ft)	Rotation (Radians)
0.000	0.00000
3.358	0.00046
6.725	0.00092
10.083	0.00138
13.592	0.00181
17.100	0.00224
20.617	0.00267
25.217	0.00337
29.817	0.00406
33.142	0.00452
36.467	0.00499
39.792	0.00545
43.225	0.00588
46.658	0.00631
50.433	0.00692
53.208	0.00801
55.558	0.00938
56.958	0.01096
58.367	0.01253
60.392	0.01470
62.417	0.01688
64.075	0.01891
65.742	0.02095
67.375	0.02299
69.408	0.02558
71.442	0.02818
73.475	0.03077
75.275	0.03320
77.083	0.03564
78.892	0.03807
80.700	0.04050
83.067	0.04323
85.433	0.04596
87.792	0.04869
90.158	0.05142
92.267	0.05439
94.383	0.05736
96.492	0.06032

Table C60 : Moment-Rotation Data : Dews - Test 3
Connection Type : Extended end-plate connection to unstiffened column

Tested by : Dews, R.J. [4.28]
Test identification: Test 3

Beam size : W16 × 26
Column size : W10 × 33
End-plate : 22.708 × 8.000 × 1.250 in.

Major parameters :	Ct=1.785in	Pt=3.500in	Lb=19.138in
	Cc=1.785in	Pc=3.750in	Li=11.638in
	G=5.500in		

Stiffeners : No
Beam fastening : Weld
Column fastening : 8 × 1.00 in. A325 bolts
Hole type : 1.0625 in. oversize holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	42.30	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : -

Remarks: 1) End-plate extended on tension and compression sides
2) Moment-rotation data provided in references [4.23,4.24]

Table C60 : Moment-Rotation Data : Dews - Test 3

Moment (kips.ft)	Rotation (Radians)
0.000	0.00000
79.850	0.00000
85.950	0.00136
91.275	0.00234
94.458	0.00337
97.642	0.00439
100.825	0.00542
104.383	0.00825
109.283	0.00941
111.175	0.01057
113.075	0.01174
115.050	0.01309
117.033	0.01445
119.017	0.01580
120.992	0.01716
122.650	0.01850
124.300	0.01985
125.958	0.02119
127.608	0.02254
129.267	0.02388
130.692	0.02527
132.117	0.02665
133.550	0.02803
134.975	0.02942

Table C61 : Moment-Rotation Data : Grundy et al. - Test T-1 Connection Type : Extended end-plate connection to unstiffened column

Tested by : Grundy, P.,Thomas, I.R. and Bennetts, I.D. [4.11]
Test identification: Test T-1

Beam size : 610 UB 113
Column size : 310 UC 240
End-plate : 30 × 12 × 1.00 in.

Major parameters :	Ct=1.500in	Pt=3.750in	Pit=0
	Cc=-	Pc=-	Pic=0
	Lb=-	G=3.500in	Pi=-

Stiffeners : No
Beam fastening : Weld
Column fastening : 10 × 7/8 in. diameter HSFG bolts
Hole type : 1.0625 in. oversize holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 775 kips.ft
Failure mode : Failure of the plate behind the butt weld

Remarks: 1) End-plate extended on tension side only
2) Two rows of four bolts in the tension region
3) Only moment-deflection curve provided , moment-rotation data derived from references [4.23,4.24]

Table C61 : Moment-Rotation Data : Grundy et al. - Test T-1

Moment (kips.ft)	Rotation (Radians)
0.000	0.00000
26.875	0.00010
53.750	0.00020
79.350	0.00030
104.950	0.00040
130.550	0.00050
156.142	0.00060
181.742	0.00070
207.734	0.00080
232.933	0.00090
258.533	0.00100
286.692	0.00110
314.850	0.00120
335.325	0.00130
355.800	0.00140
381.400	0.00155
407.000	0.00170
432.592	0.00185
458.192	0.00200
483.792	0.00215
509.383	0.00230
534.983	0.00250
560.583	0.00270
577.642	0.00287
594.708	0.00303
611.775	0.00320
627.133	0.00340
642.492	0.00360
655.292	0.00380
668.092	0.00400
670.650	0.00420
673.208	0.00440
683.450	0.00455
693.683	0.00470

Table C62 : Moment-Rotation Data : Grundy et al. - Test T-2
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Grundy, P.,Thomas, I.R. and Bennetts, I.D. [4.11]
Test identification:	Test T-2

Beam size :	610 UB 113
Column size :	310 UC 240
End-plate :	30 × 12 × 1.25 in.

Major parameters :	Ct=1.500in	Pt=3.750in	Pit=0
	Cc=-	Pc=-	Pic=0
	Lb=-	G=3.50in	Pi=-

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	10 × 7/8 in. diameter HSFG bolts
Hole type :	1.0625 in. oversize holes

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : Bolt failure

Remarks:
1) End-plate extended on tension side only
2) Two rows of four bolts in the tension region
3) Only moment-deflection curve provided , moment-rotation data derived from references [4.23,4.24]

Table C62 : Moment-Rotation Data : Grundy et al. - Test T-2

Moment (kips.ft)	Rotation (Radians)
0.000	0.00000
25.683	0.00010
51.367	0.00020
77.058	0.00030
102.742	0.00040
138.700	0.00053
174.658	0.00067
210.617	0.00080
236.300	0.00090
261.983	0.00100
287.675	0.00105
313.358	0.00110
341.608	0.00120
369.867	0.00130
398.117	0.00140
426.367	0.00150
449.483	0.00160
472.600	0.00170
495.917	0.00180
518.833	0.00190
541.950	0.00210
565.067	0.00230
588.183	0.00255
611.300	0.00280
629.283	0.00300
647.258	0.00320
657.533	0.00340
667.808	0.00360
678.083	0.00385
688.358	0.00410
693.492	0.00425
698.633	0.00440
703.767	0.00455
708.900	0.00470
714.042	0.00495
719.175	0.00520
721.750	0.00535
724.317	0.00550
721.750	0.00570
719.175	0.00590

Table C63 : Moment-Rotation Data : Johnstone and Walpole - Test T1-L
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Johnstone, N.D. and Walpole, W.R. [4.29]
Test identification:	Test T1-L

Beam size :	310 UB 46
Column size :	250 UC 89
End-plate :	570 × - × 32 mm

Major parameters :	Ct=-	Pt=178mm	Lb=-
	Cc=-	Pc=178mm	Li=-
	G=140mm		

Stiffeners :	16 mm horizontal column web stiffeners
Beam fastening :	Weld
Column fastening :	8 × M30 grade 8.8 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : -

Remarks: 1) End-plate extended on tension and compression sides
2) 16 mm web doubler plates on both sides of column web
3) Moment-rotation data provided in reference [4.24]

Table C63 : Moment-Rotation Data : Johnstone and Walpole - Test T1-L

Moment (kNm)	Rotation (Radians)
0.000	0.00000
11.38	0.00005
22.76	0.00010
34.13	0.00035
45.50	0.00060
56.88	0.00087
68.26	0.00113
79.63	0.00140
93.15	0.00135
106.66	0.00130
117.32	0.00150
127.98	0.00170
140.08	0.00230
152.17	0.00290
162.83	0.00350
173.49	0.00410
184.87	0.00465
196.25	0.00520
204.07	0.00605
211.88	0.00690
224.65	0.00970
233.22	0.01200
240.33	0.01465
247.45	0.01730
252.42	0.01950
257.40	0.02170
260.24	0.02400
263.09	0.02630
265.22	0.02860
267.35	0.03090
268.06	0.03300
268.77	0.03510
270.20	0.03870
273.04	0.04200
273.75	0.04390
274.46	0.04580
275.88	0.04940
277.30	0.05310
278.73	0.05545
280.15	0.05780
283.00	0.06130
283.71	0.06370
284.42	0.06610
285.13	0.06835

Table C64 : Moment-Rotation Data : Johnstone and Walpole - Test T1-R
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Johnstone, N.D. and Walpole, W.R. [4.29]
Test identification:	Test T1-R

Beam size :	310 UB 46
Column size :	250 UC 89
End-plate :	570 × - × 32 mm

Major parameters :	Ct=-	Pt=178mm	Lb=-
	Cc=-	Pc=178mm	Li=-
	G=140mm		

Stiffeners :	16 mm horizontal column web stiffeners
Beam fastening :	Weld
Column fastening :	8 × M30 grade 8.8 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : -

Remarks:
1) End-plate extended on tension and compression sides
2) 16 mm web doubler plates on both sides of column web
3) Moment-rotation data provided in reference [4.24]

Table C64 : Moment-Rotation Data : Johnstone and Walpole - Test T1-R

Moment (kNm)	Rotation (Radians)
0.000	0.00000
12.97	0.00012
25.94	0.00024
38.92	0.00036
51.89	0.00048
64.86	0.00060
78.41	0.00075
91.95	0.00090
104.07	0.00110
116.19	0.00130
126.16	0.00155
136.14	0.00180
151.12	0.00225
166.09	0.00270
181.06	0.00340
196.02	0.00410
206.71	0.00475
217.41	0.00540
227.39	0.00725
237.37	0.00910
251.63	0.01230
255.90	0.01430
260.18	0.01630
263.03	0.01850
265.88	0.02070
268.72	0.02315
271.58	0.02560
273.01	0.02785
274.43	0.03010
276.57	0.03225
278.72	0.03440
280.14	0.03625
281.56	0.03810
284.41	0.04035
287.73	0.04260
290.12	0.04450
292.96	0.04640
295.82	0.04870
298.67	0.05100
301.52	0.05345
304.38	0.05590
305.09	0.05810
305.80	0.06030
311.50	0.06390

Table C65 : Moment-Rotation Data : Johnstone and Walpole - Test T2-L
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Johnstone, N.D. and Walpole, W.R. [4.29]
Test identification:	Test T2-L

Beam size :	310 UB 46
Column size :	250 UC 89
End-plate :	570 × - × 32 mm

Major parameters :	Ct=-	Pt=178mm	Lb=-
	Cc=-	Pc=178mm	Li=-
	G=140mm		

Stiffeners :	16 mm horizontal column web stiffeners
Beam fastening :	Weld
Column fastening :	8 × M30 grade 8.8 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : -

Remarks:
1) End-plate extended on tension and compression sides
2) 16 mm web doubler plates on both sides of column web
3) Specimen subject to cyclic loading
3) Moment-rotation data provided in reference [4.24]

Table C65 : Moment-Rotation Data : Johnstone and Walpole - Test T2-L

Moment (kNm)	Rotation (Radians)
0.00	0.00000
13.90	0.00040
27.80	0.00080
39.67	0.00100
51.56	0.00120
63.44	0.00140
76.27	0.00180
89.10	0.00220
101.94	0.00260
113.34	0.00290
124.74	0.00320
136.14	0.00350
146.13	0.00390
156.11	0.00430
166.09	0.00470
178.91	0.00525
191.75	0.00580
204.57	0.00675
217.41	0.00770
229.53	0.01045
241.64	0.01320
247.35	0.01610
253.05	0.01900
258.76	0.02190
261.13	0.02493
263.50	0.02797
265.88	0.03100
268.72	0.03425
271.58	0.03750
272.30	0.04045
273.00	0.04340
274.43	0.04655
275.85	0.04970
278.00	0.05265
280.14	0.05560
281.09	0.05847
282.04	0.06133
282.99	0.06420
284.88	0.06687
286.80	0.06953
288.70	0.07220
290.12	0.07515
291.54	0.07810
292.96	0.08000

Table C66 : Moment-Rotation Data : Johnstone and Walpole - Test T2-R
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Johnstone, N.D. and Walpole, W.R. [4.29]
Test identification:	Test T2-R

Beam size :	310 UB 46
Column size :	250 UC 89
End-plate :	570 × - × 32 mm

Major parameters :	Ct=-	Pt=178mm	Lb=-
	Cc=-	Pc=178mm	Li=-
	G=140mm		

Stiffeners :	16 mm horizontal column web stiffeners
Beam fastening :	Weld
Column fastening :	8 × M30 grade 8.8 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : -

Remarks:
1) End-plate extended on tension and compression sides
2) 16 mm web doubler plates on both sides of column web
3) Specimen subject to cyclic loading
3) Moment-rotation data provided in reference [4.24]

Table C66 : Moment-Rotation Data : Johnstone and Walpole - Test T2-R

Moment (kNm)	Rotation (Radians)
0.00	0.00000
12.47	0.00020
25.50	0.00040
38.25	0.00060
51.01	0.00080
63.75	0.00100
73.32	0.00143
82.86	0.00187
92.41	0.00230
102.92	0.00280
113.43	0.00330
123.94	0.00380
133.49	0.00433
143.04	0.00487
152.60	0.00540
164.05	0.00595
175.51	0.00650
184.58	0.00723
193.66	0.00797
202.73	0.00870
214.92	0.01095
227.10	0.01320
232.82	0.01585
238.55	0.01850
242.14	0.02145
245.72	0.02440
245.72	0.02450
248.59	0.02740
251.45	0.03030
253.60	0.03330
255.75	0.03630
257.19	0.04000
258.61	0.04370
260.05	0.04650
261.48	0.04930
260.05	0.05230
258.61	0.05530
258.14	0.05840
257.66	0.06150
257.19	0.06460
257.90	0.06835
258.61	0.07210
260.05	0.07435
260.05	0.07435

Table C67 : Moment-Rotation Data : Johnstone and Walpole - Test T3-L
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Johnstone, N.D. and Walpole, W.R. [4.29]
Test identification:	Test T3-L

Beam size :	310 UB 46
Column size :	250 UC 89
End-plate :	570 × - × 24 mm

Major parameters :	Ct=-	Pt=102mm	Lb=-
	Cc=-	Pc=102mm	Li=-
	G=130mm		

Stiffeners :	12 mm horizontal column web stiffeners
Beam fastening :	Weld
Column fastening :	8 × M24 grade 8.8 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : -

Remarks:
1) End-plate extended on tension and compression sides
2) 12 mm web doubler plates on both sides of column web
3) Specimen subject to cyclic loading
3) Moment-rotation data provided in reference [4.24]

Table C67 : Moment-Rotation Data : Johnstone and Walpole - Test T3-L

Moment (kNm)	Rotation (Radians)
0.00	0.00000
9.47	0.00023
19.48	0.00047
29.22	0.00070
39.69	0.00087
50.14	0.00103
60.59	0.00120
70.57	0.00170
80.55	0.00220
90.52	0.00270
100.03	0.00323
109.53	0.00377
119.05	0.00430
128.55	0.00500
138.05	0.00570
147.55	0.00640
160.38	0.00785
173.22	0.00930
183.91	0.01085
194.60	0.01240
206.00	0.01560
217.41	0.01880
230.24	0.02470
233.81	0.02815
237.37	0.03160
240.21	0.03630
243.07	0.04100
245.92	0.04675
248.77	0.05250
250.90	0.05805
253.05	0.06360
253.76	0.06730
254.48	0.07100
256.76	0.07725
260.18	0.08350
261.61	0.08833
263.03	0.09317
264.45	0.09800
268.01	0.10365
171.58	0.10930
272.30	0.11405
273.00	0.11800
272.53	0.12330
272.0	60.12330

Table C68 : Moment-Rotation Data : Johnstone and Walpole - Test T3-R
Connection Type : Extended end-plate connection to stiffened column

Tested by : Johnstone, N.D. and Walpole, W.R. [4.29]
Test identification: Test T3-R

Beam size : 310 UB 46
Column size : 250 UC 89
End-plate : 570 × - × 24 mm

Major parameters :	Ct=-	Pt=102mm	Lb=-
	Cc=-	Pc=102mm	Li=-
	G=130mm		

Stiffeners : 12 mm horizontal column web stiffeners
Beam fastening : Weld
Column fastening : 8 × M24 grade 8.8 bolts
Hole type : -

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : -

Remarks: 1) End-plate extended on tension and compression sides
2) 12 mm web doubler plates on both sides of column web
3) Specimen subject to cyclic loading
3) Moment-rotation data provided in reference [4.24]

Table C68 : Moment-Rotation Data : Johnstone and Walpole - Test T3-R

Moment (kNm)	Rotation (Radians)
0.00	0.00000
18.01	0.00040
27.02	0.00060
35.07	0.00070
43.13	0.00080
51.19	0.00090
62.57	0.00135
73.95	0.00180
84.61	0.00265
95.28	0.00350
106.66	0.00420
118.03	0.00490
126.10	0.00540
134.15	0.00590
142.21	0.00640
152.17	0.00765
162.12	0.00890
172.08	0.01025
182.03	0.01160
190.56	0.01340
199.10	0.01520
205.49	0.01730
211.88	0.01940
215.21	0.02127
218.53	0.02313
221.84	0.02500
225.16	0.02733
228.49	0.02967
231.80	0.03200
234.17	0.03413
236.54	0.03627
238.90	0.03840
238.90	0.04020
238.90	0.04200
238.43	0.04360
237.97	0.04560
237.49	0.04740
238.90	0.04850

Table C69 : Moment-Rotation Data : Johnstone and Walpole - Test T4-L
Connection Type : Extended end-plate connection to stiffened column

Tested by : Johnstone, N.D. and Walpole, W.R. [4.29]
Test identification: Test T4-L

Beam size : 310 UB 46
Column size : 250 UC 89
End-plate : 570 × - × 16 mm

Major parameters :	Ct=-	Pt=115mm	Lb=-
	Cc=-	Pc=115mm	Li=-
	G=130mm		

Stiffeners : 12 mm horizontal column web stiffeners
Beam fastening : Weld
Column fastening : 8 × M24 grade 8.8 bolts
Hole type : -

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : -

Remarks:
1) End-plate extended on tension and compression sides
2) 12 mm web doubler plates on both sides of column web
3) Specimen subject to cyclic loading
3) Moment-rotation data provided in reference [4.24]

Table C69 : Moment-Rotation Data : Johnstone and Walpole - Test T4-L

Moment (kNm)	Rotation (Radians)
0.00	0.00000
11.02	0.00013
22.05	0.00027
33.06	0.00040
46.40	0.00105
59.73	0.00170
68.98	0.00223
78.22	0.00277
87.46	0.00330
99.72	0.00385
111.99	0.00440
119.98	0.00545
127.98	0.00650
137.06	0.00815
146.12	0.00980
155.18	0.01135
164.25	0.01290
173.95	0.01525
183.44	0.01760
195.18	0.02180
201.58	0.02445
207.98	0.02710
214.38	0.03055
220.78	0.03400
225.04	0.03745
229.31	0.04090
233.58	0.04475
237.84	0.04860
243.71	0.05255
249.57	0.05650
251.71	0.06015
253.84	0.06380
257.57	0.06840
260.97	0.07300
265.04	0.07700
268.77	0.08100
270.37	0.08440
271.97	0.08780
269.84	0.09260
269.84	0.09740
269.84	0.010010
269.84	0.010280

Table C70 : Moment-Rotation Data : Johnstone and Walpole - Test T4-R
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Johnstone, N.D. and Walpole, W.R. [4.29]
Test identification:	Test T4-R

Beam size :	310 UB 46
Column size :	250 UC 89
End-plate :	570 × - × 16 mm

Major parameters :	Ct=-	Pt=115mm	Lb=-
	Cc=-	Pc=115mm	Li=-
	G=130mm		

Stiffeners :	12 mm horizontal column web stiffeners
Beam fastening :	Weld
Column fastening :	8 × M24 grade 8.8 bolts
Hole type :	-

Measured Material Properties:

	Yield Stress (ksi)	Ultimate Stress (ksi)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : -

Remarks:
1) End-plate extended on tension and compression sides
2) 12 mm web doubler plates on both sides of column web
3) Specimen subject to cyclic loading
3) Moment-rotation data provided in reference [4.24]

Table C70 : Moment-Rotation Data : Johnstone and Walpole - Test T4-R

Moment (kNm)	Rotation (Radians)
0.00	0.00000
9.13	0.00060
18.25	0.00120
27.38	0.00180
36.50	0.00240
45.45	0.00293
54.40	0.00347
63.36	0.00400
71.21	0.00470
79.09	0.00540
86.95	0.00610
94.48	0.00697
102.00	0.00783
109.51	0.00870
116.66	0.00980
123.81	0.01090
130.98	0.01200
137.42	0.01263
143.86	0.01327
150.30	0.01390
157.27	0.01555
164.26	0.01720
170.15	0.01950
176.07	0.02180
179.83	0.02400
183.58	0.02620
185.36	0.02800
187.16	0.02980
188.95	0.03160
191.46	0.03393
193.96	0.03627
196.39	0.03860
197.18	0.04027
197.90	0.04193
198.61	0.04360
195.39	0.04575
192.17	0.04790

Table C71 : Moment-Rotation Data : Graham - Test P1
Connection Type : Extended end-plate connection to unstiffened column

Tested by : Graham, J. [4.31]
Test identification: Test P1

Beam size : 356 × 171 UB 45
Column size : 200 × 200 × 12 mm web UC shape
End-plate : 470 × 200 × 12 mm

Major parameters :			
	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	315.2	490.3
Beam web:	361.6	544.8
Column flange:	238.4	417.3
Column web:	-	-
End-plate:	347.6	491.6

Average pretension force of bolts : 75 kN shank tension

Failure moment: 218.3 kNm
Failure mode : Bolt fracture

Remarks:
1) End-plate extended on tension side only
2) 20 mm column flange
3) Washer used under bolt head and nut
4) Test not fully instrumented except for tightening the bolts to initial shank tension of 75 Kn
5) Moment-rotation data not recorded
6) 1200 mm beam span

Stiffeners :	No
Beam fastening :	10 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Table C72 : Moment-Rotation Data : Graham - Test P2
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Graham, J. [4.31]
Test identification:	Test P2

Beam size :	356 × 171 UB 45
Column size :	200 × 200 × 12 mm web UC shape
End-plate :	470 × 200 × 20 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners :	No
Beam fastening :	10 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	315.2	490.3
Beam web:	361.6	544.8
Column flange:	238.4	417.3
Column web:	-	-
End-plate:	238.4	417.3

Average pretension force of bolts : 75 kN shank tension

Failure moment: 196.0 kNm
Failure mode : Bolt fracture

Remarks:
1) End-plate extended on tension side only
2) 20 mm column flange
3) Washer used under bolt head and nut
4) Flange deformation and slip values were not recorded
5) 1120 mm beam span

Table C72 : Moment-Rotation Data : Graham - Test P2

Moment (kNm)	Rotation (Radians)
0.00	0.00000
20.00	0.00050
40.00	0.00070
60.00	0.00130
80.00	0.00210
100.00	0.00365
112.80	0.00502
120.00	0.00615
132.00	0.00837
140.00	0.01065
153.60	0.01674
160.00	0.02435
165.50	0.02906

Table C73 : Moment-Rotation Data : Graham - Test CS1-1
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Graham, J. [4.31]
Test identification:	Test CS1-1

Beam size :	356 × 171 UB 45
Column size :	200 × 200 × 12 mm web UC shape
End-plate :	470 × 200 × 15 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners :	No
Beam fastening :	10 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	290.7	475.7
Beam web:	290.5	464.8
Column flange:	229.0	398.4
Column web:	-	-
End-plate:	229.0	398.4

Average pretension force of bolts : 75 kN shank tension

Failure moment: 176.3 kNm

Failure mode : Bolt fracture, end-plate and column flange fracture

Remarks:
1) End-plate extended on tension side only
2) 15 mm column flange
3) Washer used under bolt head and nut
4) 1216 mm beam span

Table C73 : Moment-Rotation Data : Graham - Test CS1-1

Moment (kNm)	Rotation (Radians)
0.00	0.00000
20.00	0.00070
40.00	0.00122
48.00	0.00152
60.00	0.00259
80.00	0.00548
89.50	0.00934
100.00	0.01430
112.20	0.02510
120.00	0.03255
130.20	0.04535
140.00	0.05465
151.00	0.07320

Table C74 : Moment-Rotation Data : Graham - Test CS1-2 Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Graham, J. [4.31]
Test identification:	Test CS1-2

Beam size :	356 × 171 UB 45
Column size :	200 × 200 × 12 mm web UC shape
End-plate :	470 × 200 × 15 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners :	No
Beam fastening :	10 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	290.7	475.7
Beam web:	290.5	464.8
Column flange:	229.0	398.4
Column web:	-	-
End-plate:	229.0	398.4

Average pretension force of bolts : 75 kN shank tension

Failure moment: 164.4 kNm
Failure mode : Bolt fracture

Remarks:
1) End-plate extended on tension side only
2) 15 mm column flange
3) Washer used under bolt head and nut
4) 1015 mm beam span

Table C74 : Moment-Rotation Data : Graham - Test CS1-2

Moment (kNm)	Rotation (Radians)
0.00	0.00000
24.60	0.00075
40.00	0.00135
49.20	0.00253
52.80	0.00300
60.00	0.00374
76.80	0.00820
80.00	0.00983
89.40	0.01400
100.00	0.02130
109.20	0.02570
117.00	0.03195
127.80	0.04596
139.20	0.06970
156.00	0.06852

Table C75 : Moment-Rotation Data : Graham - Test CS1-3 Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Graham, J. [4.31]
Test identification:	Test CS1-3

Beam size :	356 × 171 UB 45
Column size :	200 × 200 × 12 mm web UC shape
End-plate :	470 × 200 × 15 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners :	No
Beam fastening :	10 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	290.7	475.7
Beam web:	290.5	465.8
Column flange:	229.0	398.4
Column web:	-	-
End-plate:	229.0	398.4

Average pretension force of bolts : 75 kN shank tension

Failure moment: 160.5 kNm
Failure mode : Column flange fracture

Remarks:
1) End-plate extended on tension side only
2) 15 mm column flange
3) Washer used under bolt head and nut
4) 810 mm beam span

Table C75 : Moment-Rotation Data : Graham - Test CS1-3

Moment (kNm)	Rotation (Radians)
0.00	0.00000
20.00	0.00042
40.00	0.00075
52.80	0.00129
60.00	0.00210
67.20	0.00345
80.00	0.00630
91.20	0.00990
100.00	0.01620
112.80	0.02310
120.00	0.03120
134.50	0.04260

Table C76 : Moment-Rotation Data : Graham - Test CS1-4
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Graham, J. [4.31]
Test identification:	Test CS1-4

Beam size :	356 × 171 UB 45
Column size :	200 × 200 × 12 mm web UC shape
End-plate :	470 × 200 × 15 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners :	No
Beam fastening :	10 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	290.7	475.7
Beam web:	290.5	464.8
Column flange:	229.0	398.4
Column web:	-	-
End-plate:	229.0	398.4

Average pretension force of bolts : 75 kN shank tension

Failure moment:	167.5 kNm
Failure mode :	Bolt fracture

Remarks:
1) End-plate extended on tension side only
2) 15 mm column flange
3) Washer used under bolt head and nut
4) 565 mm beam span

Table C76 : Moment-Rotation Data : Graham - Test CS1-4

Moment (kNm)	Rotation (Radians)
0.00	0.00000
28.80	0.00060
40.00	0.00105
57.60	0.00270
71.40	0.00480
80.00	0.00720
87.00	0.00930
100.00	0.01560
114.00	0.02310
120.00	0.02760
136.20	0.04200
140.00	0.04922

Table C77 : Moment-Rotation Data : Graham - Test CS1-5 Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Graham, J. [4.31]
Test identification:	Test CS1-5

Beam size :	356 × 171 UB 45
Column size :	200 × 200 × 12 mm web UC shape
End-plate :	470 × 200 × 15 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners :	No
Beam fastening :	10 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	290.7	475.7
Beam web:	290.5	464.8
Column flange:	229.0	398.4
Column web:	-	-
End-plate:	229.0	398.4

Average pretension force of bolts : 75 kN shank tension

Failure moment: (Max moment 135.8 kNm)
Failure mode : Test discontinued due to damage to hydraulic ram

Remarks: 1) End-plate extended on tension side only
2) 15 mm column flange
3) Washer used under bolt head and nut
4) 315 mm beam span

Table C77 : Moment-Rotation Data : Graham - Test CS1-5

Moment (kNm)	Rotation (Radians)
0.00	0.00000
8.00	0.00030
16.20	0.00045
24.00	0.00051
30.00	0.00075
38.40	0.00105
48.00	0.00120
55.20	0.00189
63.60	0.00300

Table C78 : Moment-Rotation Data : Graham - Test CS2-1
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Graham, J. [4.31]
Test identification:	Test CS2-1

Beam size :	356 × 171 UB 45
Column size :	200 × 200 × 12 mm web UC shape
End-plate :	470 × 200 × 15 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners :	No
Beam fastening :	8 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	290.7	475.5
Beam web:	290.5	464.8
Column flange:	229.0	398.4
Column web:	-	-
End-plate:	229.0	398.4

Average pretension force of bolts : 75 kN shank tension

Failure moment: 163.9 kNm
Failure mode : Bolt fracture and column flange fracture

Remarks:
1) End-plate extended on tension side only
2) 11.8 mm column flange
3) Washer used under bolt head and nut
4) 1218 mm beam span

Table C78 : Moment-Rotation Data : Graham - Test CS2-1

Moment (kNm)	Rotation (Radians)
0.00	0.00000
9.60	0.00030
15.60	0.00060
24.00	0.00105
31.20	0.00180
38.40	0.00261
47.70	0.00420
53.40	0.00600
69.60	0.01140
76.80	0.01515
85.20	0.02040
93.60	0.02650
101.40	0.03360
108.00	0.04095
122.40	0.05700

Table C79 : Moment-Rotation Data : Graham - Test CS2-2 Connection Type : Extended end-plate connection to unstiffened column

Tested by : Graham, J. [4.31]
Test identification: Test CS2-2

Beam size : 356 × 171 UB 45
Column size : 200 × 200 × 12 mm web UC shape
End-plate : 470 × 200 × 15 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners : No
Beam fastening : 8 mm fillet weld
Column fastening : 6 × M16 HSFG bolts
Hole type : 18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	290.7	475.5
Beam web:	290.5	464.8
Column flange:	229.0	398.4
Column web:	-	-
End-plate:	229.0	398.4

Average pretension force of bolts : 75 kN shank tension

Failure moment: 163.9 kNm
Failure mode : Bolt fracture

Remarks: 1) End-plate extended on tension side only
2) 11.95 mm column flange
3) Washer used under bolt head and nut
4) 1010 mm beam span

Table C79 : Moment-Rotation Data : Graham - Test CS2-2

Moment (kNm)	Rotation (Radians)
0.00	0.00000
20.00	0.00067
30.50	0.00113
40.00	0.00183
49.80	0.00304
60.00	0.00545
74.40	0.01095
80.00	0.01430
89.40	0.01932
100.00	0.02800
110.40	0.03926
120.00	0.04610
129.60	0.06178
140.00	0.06765

Table C80 : Moment-Rotation Data : Graham - Test CS2-3 Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Graham, J. [4.31]
Test identification:	Test CS2-3

Beam size :	356 × 171 UB 45
Column size :	200 × 200 × 12 mm web UC shape
End-plate :	470 × 200 × 15 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners :	No
Beam fastening :	8 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	290.7	475.5
Beam web:	290.5	464.8
Column flange:	229.0	398.4
Column web:	-	-
End-plate:	229.0	398.4

Average pretension force of bolts : 75 kN shank tension

Failure moment: 164.5 kNm
Failure mode : Column flange fracture

Remarks:
1) End-plate extended on tension side only
2) 12 mm column flange
3) Washer used under bolt head and nut
4) 815 mm beam span

Table C80 : Moment-Rotation Data : Graham - Test CS2-3

Moment (kNm)	Rotation (Radians)
0.00	0.00000
10.20	0.00030
20.00	0.00060
29.40	0.00150
40.80	0.00300
50.40	0.00510
60.60	0.00840
72.00	0.01215
82.80	0.01860
92.40	0.02640
103.20	0.03480
114.00	0.04500
122.40	0.05456

Table C81 : Moment-Rotation Data : Graham - Test CS2-4
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Graham, J. [4.31]
Test identification:	Test CS2-4

Beam size :	356 × 171 UB 45
Column size :	200 × 200 × 12 mm web UC shape
End-plate :	470 × 200 × 15 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners :	No
Beam fastening :	8 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	290.7	475.5
Beam web:	290.5	464.8
Column flange:	229.0	398.4
Column web:	-	-
End-plate:	229.0	398.4

Average pretension force of bolts : 75 kN shank tension

Failure moment: 159.1 kNm
Failure mode : Bolt fracture and column flange fracture

Remarks:
1) End-plate extended on tension side only
2) 11.74 mm column flange
3) Washer used under bolt head and nut
4) 565 mm beam span

Table C81 : Moment-Rotation Data : Graham - Test CS2-4

Moment (kNm)	Rotation (Radians)
0.00	0.00000
14.40	0.00042
21.60	0.00060
27.60	0.00090
35.40	0.00150
42.00	0.00210
48.60	0.00330
56.40	0.00480
67.20	0.00720
70.80	0.01020
86.40	0.01980
98.40	0.02910
114.00	0.04200
127.20	0.05580

Table C82 : Moment-Rotation Data : Graham - Test CS2-5 Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Graham, J. [4.31]
Test identification:	Test CS2-5

Beam size :	356 × 171 UB 45
Column size :	200 × 200 × 12 mm web UC shape
End-plate :	470 × 200 × 15 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners :	No
Beam fastening :	8 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	290.7	475.5
Beam web:	290.5	464.8
Column flange:	229.0	398.4
Column web:	-	-
End-plate:	229.0	398.4

Average pretension force of bolts : 75 kN shank tension

Failure moment: (Max moment 106.6 kNm)
Failure mode : Test discontinued due to damage to hydraulic ram

Remarks:
1) End-plate extended on tension side only
2) 11.6 mm column flange
3) Washer used under bolt head and nut
4) 317 mm beam span

Table C82 : Moment-Rotation Data : Graham - Test CS2-5

Moment (kNm)	Rotation (Radians)
0.00	0.00000
8.40	0.00030
15.60	0.00057
24.00	0.00075
31.20	0.00129
40.60	0.00210
46.80	0.00360
55.20	0.00585
62.40	0.00900
70.80	0.01209
79.20	0.01830
86.40	0.02700
91.20	0.03300
96.00	0.03900
102.00	0.04740

Table C83 : Moment-Rotation Data : Graham - Test CS3-1
Connection Type : Extended end-plate connection to unstiffened column

Tested by : Graham, J. [4.31]
Test identification: Test CS3-1

Beam size : 357 × 171 UB 45
Column size : 200 × 200 × 12 mm web UC shape
End-plate : 470 × 200 × 20 mm

Major parameters :			
	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners : No
Beam fastening : 10 mm fillet weld
Column fastening : 6 × M16 HSFG bolts
Hole type : 18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	290.7	475.5
Beam web:	290.5	464.8
Column flange:	328.1	503.1
Column web:	-	-
End-plate:	328.1	503.1

Average pretension force of bolts : 75 kN shank tension

Failure moment: 197.0 kNm
Failure mode : Bolt fracture

Remarks:
1) End-plate extended on tension side only
2) 20 mm column flange
3) Washer used under bolt head and nut
4) 1220 mm beam span

Table C83 : Moment-Rotation Data : Graham - Test CS3-1

Moment (kNm)	Rotation (Radians)
0.00	0.00000
15.60	0.00030
30.60	0.00069
45.60	0.00120
62.40	0.00150
76.80	0.00240
92.40	0.00315
108.00	0.00390
123.60	0.00510
153.60	0.00810
169.20	0.01095
184.80	0.01530

Table C84 : Moment-Rotation Data : Graham - Test CS3-2 Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Graham, J. [4.31]
Test identification:	Test CS3-2

Beam size :	356 × 171 UB 45
Column size :	200 × 200 × 12 mm web UC shape
End-plate :	470 × 200 × 20 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners :	No
Beam fastening :	10 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	290.7	475.5
Beam web:	290.5	464.8
Column flange:	328.1	503.1
Column web:	-	-
End-plate:	328.1	503.1

Average pretension force of bolts : 75 kN shank tension

Failure moment: 193.1 kNm
Failure mode : Bolt fracture

Remarks: 1) End-plate extended on tension side only
2) 20 mm column flange
3) Washer used under bolt head and nut
4) 1015 mm beam span

Table C84 : Moment-Rotation Data : Graham - Test CS3-2

Moment (kNm)	Rotation (Radians)
0.00	0.00000
20.00	0.00040
40.00	0.00075
60.00	0.00137
80.00	0.00213
100.00	0.00313
120.00	0.00455
140.00	0.00636
150.60	0.00760
160.00	0.00904
176.20	0.01252

Table C85 : Moment-Rotation Data : Graham - Test CS3-3 Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Graham, J. [4.31]
Test identification:	Test CS3-3

Beam size :	356 × 171 UB 45
Column size :	200 × 200 × 12 mm web UC shape
End-plate :	470 × 200 × 20 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners :	No
Beam fastening :	10 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	290.7	475.5
Beam web:	290.5	464.8
Column flange:	328.1	503.1
Column web:	-	-
End-plate:	328.1	503.1

Average pretension force of bolts : 75 kN shank tension

Failure moment: 190.0 kNm
Failure mode : Bolt fracture

Remarks:
1) End-plate extended on tension side only
2) 20 mm column flange
3) Washer used under bolt head and nut
4) 820 mm beam span

Table C85 : Moment-Rotation Data : Graham - Test CS3-3

Moment (kNm)	Rotation (Radians)
0.00	0.00000
20.00	0.00030
41.40	0.00075
62.40	0.00129
82.80	0.00189
103.20	0.00330
124.80	0.00480
145.20	0.00630
154.80	0.00750
166.80	0.00900
174.00	0.01159

Table C86 : Moment-Rotation Data : Graham - Test CS3-4
Connection Type : Extended end-plate connection to unstiffened column

Tested by : Graham, J. [4.31]
Test identification: Test CS3-4

Beam size : 356 × 171 UB 45
Column size : 200 × 200 × 12 mm web UC shape
End-plate : 470 × 200 × 20 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners : No
Beam fastening : 10 mm fillet weld
Column fastening : 6 × M16 HSFG bolts
Hole type : 18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	290.7	475.5
Beam web:	290.5	464.8
Column flange:	328.1	503.1
Column web:	-	-
End-plate:	328.1	503.1

Average pretension force of bolts : 75 kN shank tension

Failure moment: 194.9 kNm
Failure mode : Bolt fracture

Remarks: 1) End-plate extended on tension side only
2) 20 mm column flange
3) Washer used under bolt head and nut
4) 571 mm beam span

Table C86 : Moment-Rotation Data : Graham - Test CS3-4

Moment (kNm)	Rotation (Radians)
0.00	0.00000
27.60	0.00030
45.60	0.00069
57.60	0.00120
72.00	0.00180
86.40	0.00240
100.80	0.00345
115.20	0.00435
122.40	0.00510
129.60	0.00570
144.00	0.00690
158.40	0.00900
170.50	0.01148

Table C87 : Moment-Rotation Data : Graham - Test CS3-5
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Graham, J. [4.31]
Test identification:	Test CS3-5

Beam size :	356 × 171 UB 45
Column size :	200 × 200 × 12 mm web UC shape
End-plate :	470 × 200 × 20 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners :	No
Beam fastening :	10 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	290.7	475.5
Beam web:	290.5	464.8
Column flange:	328.1	503.1
Column web:	-	-
End-plate:	328.1	503.1

Average pretension force of bolts : 75 kN shank tension

Failure moment: (Max moment 138.1 kNm)
Failure mode : Test discontinued due to damage to hydraulic ram

Remarks:
1) End-plate extended on tension side only
2) 20 mm column flange
3) Washer used under bolt head and nut
4) 326 mm beam span

Table C87 : Moment-Rotation Data : Graham - Test CS3-5

Moment (kNm)	Rotation (Radians)
0.00	0.00000
8.40	0.00030
16.20	0.00060
24.00	0.00090
33.00	0.00090
40.80	0.00165
50.00	0.00915
57.60	0.00240
65.40	0.00270
74.40	0.00360
81.60	0.00390
91.20	0.00480
100.00	0.00540
108.00	0.00675
115.20	0.01020
122.40	0.01695
125.40	0.01950

Table C88 : Moment-Rotation Data : Graham - Test CS4-1
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Graham, J. [4.31]
Test identification:	Test CS4-1

Beam size :	356 × 171 UB 45
Column size :	200 × 200 × 12 mm web UC shape
End-plate :	470 × 200 × 15 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners :	No
Beam fastening :	8 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	304.2	460.9
Beam web:	315.1	467.9
Column flange:	328.1	503.1
Column web:	-	-
End-plate:	328.1	503.1

Average pretension force of bolts : 75 kN shank tension

Failure moment: 163.9 kNm
Failure mode : Bolt fracture

Remarks: 1) End-plate extended on tension side only
2) 20 mm column flange
3) Washer used under bolt head and nut
4) 1218 mm beam span

Table C88 : Moment-Rotation Data : Graham - Test CS4-1

Moment (kNm)	Rotation (Radians)
0.00	0.00000
20.00	0.00045
40.00	0.00120
60.00	0.00243
80.00	0.00395
88.80	0.00487
100.00	0.00608
110.40	0.00675
120.00	0.00797
129.60	0.00944
140.00	0.01126
147.00	0.01216

Table C89 : Moment-Rotation Data : Graham - Test CS4-3
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Graham, J. [4.31]
Test identification:	Test CS4-3

Beam size :	356 × 171 UB 45
Column size :	200 × 200 × 12 mm web UC shape
End-plate :	470 × 200 × 15 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners :	No
Beam fastening :	8 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	304.2	460.9
Beam web:	315.1	467.9
Column flange:	328.1	503.1
Column web:	-	-
End-plate:	328.1	503.1

Average pretension force of bolts : 75 kN shank tension

Failure moment: 166.5 kNm
Failure mode : Bolt fracture

Remarks:
1) End-plate extended on tension side only
2) 20 mm column flange
3) Washer used under bolt head and nut
4) 815 mm beam span

Table C89 : Moment-Rotation Data : Graham - Test CS4-3

Moment (kNm)	Rotation (Radians)
0.00	0.00000
20.40	0.00030
40.80	0.00105
61.20	0.00180
74.40	0.00270
91.20	0.00390
101.40	0.00495
112.80	0.00600
122.40	0.00720
132.00	0.00900
143.40	0.01140
148.80	0.01290
153.60	0.01395
158.40	0.01680
163.20	0.01957

Table C90 : Moment-Rotation Data : Graham - Test CS5-1 Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Graham, J. [4.31]
Test identification:	Test CS5-1

Beam size :	356 × 171 UB 45
Column size :	200 × 200 × 12 mm web UC shape
End-plate :	470 × 200 × 20 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners :	No
Beam fastening :	10 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	304.2	460.9
Beam web:	315.1	467.9
Column flange:	328.1	503.1
Column web:	-	-
End-plate:	328.1	503.1

Average pretension force of bolts : 75 kN shank tension

Failure moment: 210.3 kNm
Failure mode : Bolt fracture

Remarks:
1) End-plate extended on tension side only
2) 17.5 mm column flange
3) Washer used under bolt head and nut
4) 1023 mm beam span

Table C90 : Moment-Rotation Data : Graham - Test CS5-1

Moment (kNm)	Rotation (Radians)
0.00	0.00000
20.00	0.00045
40.00	0.00122
60.00	0.00183
75.00	0.00295
86.40	0.00347
100.00	0.00487
117.00	0.00570
130.80	0.00730
141.60	0.00943
159.60	0.01338
176.40	0.01917
184.20	0.02381

Table C91 : Moment-Rotation Data : Graham - Test CS5-2 Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Graham, J. [4.31]
Test identification:	Test CS5-2

Beam size :	356 × 171 UB 45
Column size :	200 × 200 × 12 mm web UC shape
End-plate :	470 × 200 × 20 mm

Major parameters :	Ct=30mm	Pt=110mm	Pit=-
	Cc=38mm	Pc=50mm	Pic=-
	Lb=342mm	G=140mm	Pi=232mm

Stiffeners :	No
Beam fastening :	10 mm fillet weld
Column fastening :	6 × M16 HSFG bolts
Hole type :	18 mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	304.2	460.9
Beam web:	315.1	467.9
Column flange:	328.1	503.1
Column web:	-	-
End-plate:	328.1	503.1

Average pretension force of bolts : 75 kN shank tension

Failure moment: (max moment 155.5 kNm)
Failure mode : Test discontinued due to damage to hydraulic ram

Remarks:
1) End-plate extended on tension side only
2) 17 mm column flange
3) Washer used under bolt head and nut
4) 320 mm beam span

Table C91 : Moment-Rotation Data : Graham - Test CS5-2

Moment (kNm)	Rotation (Radians)
0.00	0.00000
16.20	0.00030
34.80	0.00060
50.40	0.00180
64.80	0.00240
81.60	0.00330
98.40	0.00510
105.60	0.00840
114.00	0.01320
122.40	0.02280
129.60	0.03120
139.20	0.03880

Table C92 : Moment-Rotation Data : Moore and Sims - Tests J1 Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Moore, D.B. and Sims, P.A.C. [4.35]
Test identification:	Test J1

Beam size :	254 × 102 × 22 UB
Column size :	152 × 152 × 23 UC
End-plate :	355 × 147 × 15 mm

Major parameters :	Ct=40mm	Pt=100mm	Pit=0
	Cc+Pc=80mm	Pc=-	Pic=0
	Lb=235mm	G=87mm	Pi=135mm

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	6 × M16 grade 8.8 bolts
Hole type :	18mm clerance holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	309	-
Column web:	309	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment: 76.9kNm
Failure mode : Bolt failure

Remarks: 1) Moment-rotation not provided
2) End-plate extended on tension side only

Table C93 : Moment-Rotation Data : Moore and Sims - Tests J2 Connection Type : Extended end-plate connection to stiffened column

Tested by :	Moore, D.B. and Sims, P.A.C. [4.35]
Test identification:	Test J2

Beam size :	254 × 102 × 22 UB
Column size :	152 × 152 × 23 UC
End-plate :	355 × 147 × 15 mm

Major parameters :	Ct=40mm	Pt=100mm	Pit=0
	Cc+Pc=80mm	Pc=-	Pic=0
	Lb=235mm	G=87mm	Pi=135mm

Stiffeners :	Backing plates 250× 66 × 6mm
Beam fastening :	Weld
Column fastening :	6 × M16 grade 8.8 bolts
Hole type :	18mm clerance holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	309	-
Column web:	309	-
End-plate:	-	-
Backing plate:	292	-

Average pretension force of bolts : -

Failure moment: 81.4kNm
Failure mode : Web and flange fracture

Remarks: 1) Moment-rotation not provided
2) End-plate extended on tension side only

Table C94 : Moment-Rotation Data : Moore and Sims - Test J3 Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Moore, D.B. and Sims, P.A.C. [4.35]
Test identification:	Test J3

Beam size :	254 × 102 × 22 UB
Column size :	152 × 152 × 23 UC
End-plate :	355 × 147 × 15 mm

Major parameters :	Ct=40mm	Pt=100mm	Pit=0
	Cc+Pc=80mm	Pc=-	Pic=0
	Lb=235mm	G=87mm	Pi=135mm

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	6 × M16 grade 8.8 bolts
Hole type :	18mm clerance holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	309	-
Column web:	309	-
End-plate:	-	-

Average pretension force of bolts : -

Failure moment:	76.5kNm
Failure mode :	Web and flange fracture followed by bolt failure

Remarks: 1) Moment-rotation data derived from M-O curve
2) End-plate extended on tension side only

Table C94 : Moment-Rotation Data : Moore and Sims - Tests J3

Moment (kNm)	Rotation (radians)
0.00	0.00000
5.90	0.00125
9.80	0.00200
13.00	0.00270
17.60	0.00345
20.00	0.00420
25.00	0.00580
28.40	0.00740
31.70	0.00965
35.90	0.01330
38.80	0.01730
40.00	0.01875
41.75	0.02170
43.00	0.02500
45.30	0.02860
48.60	0.03550
51.50	0.04100
54.80	0.04740
56.10	0.05000
58.00	0.05330
60.00	0.05625
62.60	0.06170
67.20	0.07010
69.50	0.07500
72.70	0.08140
76.80	0.09230

Table C95 : Moment-Rotation Data : Moore and Sims - Tests J4 Connection Type : Extended end-plate connection to stiffened column

Tested by : Moore, D.B. and Sims, P.A.C. [4.35]
Test identification: Test J4

Beam size : 254 × 102 × 22 UB
Column size : 152 × 152 × 23 UC
End-plate : 355 × 147 × 15 mm

Major parameters :			
	Ct=40mm	Pt=100mm	Pit=0
	Cc+Pc=80mm	Pc=-	Pic=0
	Lb=235mm	G=87mm	Pi=135mm

Stiffeners : Backing plate 250 × 66 × 6mm
Beam fastening : Weld
Column fastening : 6 × M16 grade 8.8 bolts
Hole type : 18mm clerance holes

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	309	-
Column web:	309	-
End-plate:	-	-
Backing plate:	292	-

Average pretension force of bolts : -

Failure moment: 85.9kNm
Failure mode : Beam and end-plate weld fracture

Remarks: 1) Moment-rotation data derived from M-O curve
2) End-plate extended on tension side only

Table C95 : Moment-Rotation Data : Moore and Sims - Test J4

Moment (kNm)	Rotation (radians)
0.00	0.00000
6.20	0.00100
9.50	0.00150
13.40	0.00220
17.60	0.00300
20.00	0.00345
24.80	0.00420
30.00	0.00570
34.00	0.00715
37.80	0.00890
40.00	0.00990
41.75	0.01085
47.60	0.01405
53.50	0.01825
60.00	0.02295
61.50	0.02500
64.60	0.02765
70.45	0.03420
74.74	0.04050

Table C96 : Moment-Rotation Data : Zoetemeijer and Munter - Test 1
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P. and Munter, H. [4.36]
Test identification:	Test 1

Beam size :	IPE 400
Column size :	HE 240A
End-plate :	500 × 240 × 20.5 mm

Major parameters :	Ct=40mm	Pt=110mm	Pit=0
	Cc=-	Pc=-	Pic=0
	Lb=385mm	G=110mm	Pi=275mm

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	6 × M20 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	241	-
Column web:	-	-
End-plate:	232	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : -

Remarks: 1) Moment-rotation data derived from M-O curve
2) No axial load in column

Table C96 : Moment-Rotation Data : Zoetemeijer and Munter - Test 1

Moment (kNm)	Rotation (radians)
0.00	0.00000
15.70	0.00035
31.40	0.00070
54.30	0.00140
64.30	0.00210
87.10	0.00280
100.00	0.00490
115.70	0.00840
128.60	0.01120
137.15	0.01540
142.90	0.01890
147.10	0.02170
148.60	0.02415
161.40	0.02940
163.60	0.03290
170.00	0.03605
171.40	0.04130
180.00	0.04550
182.90	0.05110
191.40	0.05530
194.30	0.05950
195.70	0.06525
200.00	0.07000
205.70	0.07735
211.40	0.08505
214.30	0.08960
215.70	0.08960
218.60	0.09870
221.40	0.10290
222.90	0.10570

Table C97 : Moment-Rotation Data : Zoetemeijer and Munter - Test 2
Connection Type : Extended end-plate connection to unstiffened column

Tested by : Zoetemeijer, P. and Munter, H. [4.36]
Test identification: Test 2

Beam size : IPE 400
Column size : HE 240A
End-plate : 500 × 240 × 20.5 mm

Major parameters :	Ct=40mm	Pt=110mm	Pit=0
	Cc=-	Pc=-	Pic=0
	Lb=385mm	G=110mm	Pi=275mm

Stiffeners : No
Beam fastening : Weld
Column fastening : 6 × M20 grade 8.8 bolts
Hole type : -

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	241	-
Column web:	-	-
End-plate:	232	-

Average pretension force of bolts : -

Failure moment: Max. recorded moment 211.4kNm
Failure mode : -

Remarks: 1) Moment-rotation data derived from M-O curve
2) 135(N/mm^2) stress incolumn flanges

Table C98 : Moment-Rotation Data : Zoetemeijer and Munter - Test 2

Moment (kNm)	Rotation (radians)
0.00	0.00000
22.85	0.00070
44.30	0.00105
68.60	0.00140
85.70	0.00210
111.40	0.00420
125.70	0.00665
134.30	0.00094
141.40	0.00126
148.60	0.01680
152.85	0.02030
157.15	0.02450
158.60	0.02765
160.70	0.03290
162.85	0.03780
168.60	0.04095
171.40	0.04410
174.30	0.04690
172.85	0.05250
180.00	0.05460
182.90	0.05880
185.70	0.06440
190.00	0.06895
188.60	0.07210
188.60	0.07560
197.15	0.07980
197.85	0.08575
197.15	0.08890
197.15	0.09100
201.40	0.09310
200.00	0.09590
203.60	0.09835
211.40	0.09975
206.40	0.10220
210.00	0.10290
205.70	0.10500

Table C98 : Moment-Rotation Data : Zoetemeijer and Munter - Test 3
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P. and Munter, H. [4.36]
Test identification:	Test 3

Beam size :	IPE 400
Column size :	HE 240A
End-plate :	500 × 240 × 20.5 mm

Major parameters :	Ct=40mm	Pt=110mm	Pit=0
	Cc=-	Pc=-	Pic=0
	Lb=385mm	G=110mm	Pi=275mm

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	6 × M20 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	241	-
Column web:	-	-
End-plate:	232	-

Average pretension force of bolts : -

Failure moment: -
Failure mode : -

Remarks: 1) Moment-rotation data derived from M-O curve
2) 143(N/mm^2) stress increasing to 235(N/mm^2) in column flanges

Table C98 : Moment-Rotation Data : Zoetemeijer and Munter - Test 3

Moment (kNm)	Rotation (radians)
0.00	0.00000
20.80	0.00595
23.60	0.01019
45.80	0.02038
57.60	0.03057
70.80	0.04076
81.60	0.05265
94.40	0.06793
104.15	0.08490
111.10	0.09850
115.30	0.11548
120.15	0.01308
115.30	0.01359
121.50	0.14690
118.00	0.15284
112.50	0.15624
115.30	0.16302
108.30	0.16813
102.80	0.17320
99.80	0.17662
101.40	0.18511
107.00	0.19700
111.10	0.20973
111.50	0.22247
111.80	0.23436
112.50	0.24964
114.60	0.26832
118.00	0.28955
118.00	0.30569
113.90	0.32946
110.40	0.33625

Table C99 : Moment-Rotation Data : Zoetemeijer and Munter - Test 4
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Zoetemeijer, P. and Munter, H. [4.36]
Test identification:	Test 4

Beam size :	IPE 400
Column size :	HE 240A
End-plate :	500 × 240 × 20.5 mm

Major parameters :	Ct=40mm	Pt=110mm	Pit=0
	Cc=-	Pc=-	Pic=0
	Lb=385mm	G=110mm	Pi=275mm

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	6 × M20 grade 8.8 bolts
Hole type :	-

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	241	-
Column web:	-	-
End-plate:	232	-

Average pretension force of bolts : -

Failure moment: Max. recorded moment 171.6kNm
Failure mode : -

Remarks: 1) Moment-rotation data derived from M-O curve
2) 73(N/mm^2) stress increasing to 233(N/mm^2) in column flanges

Table C99 : Moment-Rotation Data : Zoetemeijer and Munter - Test 4

Moment (kNm)	Rotation (radians)
0.00	0.00000
22.30	0.00028
41.85	0.00083
55.80	0.00125
61.40	0.00181
84.40	0.00250
89.30	0.00278
100.50	0.00340
115.80	0.03900
135.35	0.00403
114.40	0.00528
133.95	0.00556
129.80	0.00834
114.40	0.00890
134.00	0.00945
126.30	0.01029
125.60	0.01237
115.80	0.01293
131.15	0.01418
140.25	0.01515
125.60	0.01682
145.10	0.01835
152.10	0.02100
153.50	0.02294
141.60	0.02530
160.50	0.02753
150.70	0.03365
145.10	0.03643
153.50	0.03865
167.40	0.04170
171.60	0.05060

Table C100 : Moment-Rotation Data : Tong - Test 2
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Tong, C. S. [4.37]
Test identification:	Test 2

Beam size :	305 × 165 UB 40
Column size :	254 × 254 UC 89
End-plate :	400mm × 200mm × 12mm

Major parameters :	Ct=35mm	Pt=111mm	Pit=0mm
	Cc+Pc=100mm	Pc= -	Pic=0mm
	Lb=265mm	G=125mm	Pi=154mm

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	22mm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	300	-

Average pretension force of bolts : 25 kN

Failure moment: 98.0 kNm
Failure mode : weld at tension flange

Remarks: 1) End-plate extended on tension side only
2) Pure moment loading

Table C100 : Moment-Rotation Data : Tong - Test 2

Moment (kNm)	Rotation (Radians)
0.00	0.00000
18.00	0.00050
28.00	0.00080
38.00	0.00120
48.50	0.00190
58.50	0.00250
64.50	0.00310
70.00	0.00390
74.00	0.00470
80.50	0.00570
84.50	0.00700
92.50	0.00960

Table C101 : Moment-Rotation Data : Tong - Test 4
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Tong, C. S. [4.37]
Test identification:	Test 4

Beam size :	305 × 165 UB 54
Column size :	254 × 254 UC 132
End-plate :	440mm × 240mm × 12mm

Major parameters :	Ct=60mm	Pt=120mm	Pit=0mm
	Cc+Pc=85mm	Pc=-	Pic=0mm
	Lb=295mm	G=120mm	Pi=175mm

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	22mm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	300	-

Average pretension force of bolts : 25 kN

Failure moment: 190.0 kNm
Failure mode : weld at tension flange

Remarks: 1) End-plate extended on tension side only
2) Pure moment loading

Table C101 : Moment-Rotation Data : Tong - Test 4

Moment (kNm)	Rotation (Radians)
0.00	0.00000
18.90	0.00060
40.00	0.00130
58.75	0.00240
78.65	0.00350
99.55	0.00550
108.50	0.00750
119.45	0.01040
130.40	0.01340
135.85	0.01550
140.00	0.01760
145.35	0.02000
149.30	0.02290
155.25	0.02640
160.25	0.02960
164.25	0.03460

Table C102 : Moment-Rotation Data : Tong - Test 6
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Tong, C. S. [4.37]
Test identification:	Test 6

Beam size :	305 × 165 UB 54
Column size :	254 × 254 UC 73
End-plate :	440mm × 240mm × 25mm

Major parameters :	Ct=60mm	Pt=120mm	Pit=0mm
	Cc+Pc=85mm	Pc=-	Pic=0mm
	Lb=295mm	G=120mm	Pi=175mm

Stiffeners :	No
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	22mm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 25 kN

Failure moment: 100.0 kNm
Failure mode : Column web buckled

Remarks: 1) End-plate extended on tension side only
2) Pure moment loading

Table C102 : Moment-Rotation Data : Tong - Test 6

Moment (kNm)	Rotation (Radians)
0.00	0.00000
10.30	0.00030
20.80	0.00060
30.00	0.00120
40.00	0.00180
50.00	0.00250
60.00	0.00320
69.75	0.00400
80.00	0.00490
90.30	0.00690

Table C103 : Moment-Rotation Data : Tong - Test 7
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Tong, C. S. [4.37]
Test identification:	Test 7

Beam size :	305 × 165 UB 54
Column size :	254 × 254 UC 132
End-plate :	440mm × 240mm × 15mm

Major parameters :	Ct=60mm	Pt=120mm	Pit=0mm
	Cc+Pc=85mm	Pc=-	Pic=0mm
	Lb=295mm	G=120mm	Pi=175mm

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	22mm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 25 kN

Failure moment: (Maximum recorded moment 185.0kNm)
Failure mode : Failure stage not reached

Remarks: 1) End-plate extended on tension side only
2) Pure moment loading

Table C103 : Moment-Rotation Data : Tong - Test 7

Moment (kNm)	Rotation (Radians)
0.00	0.00000
20.05	0.00040
40.00	0.00090
50.00	0.00120
69.75	0.00210
80.75	0.00250
90.00	0.00300
99.80	0.00350
110.00	0.00400
120.90	0.00480
130.20	0.00550
150.25	0.00740
169.80	0.01014
180.30	0.01320
185.30	0.01510

Table C104 : Moment-Rotation Data : Tong - Test 8
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Tong, C. S. [4.37]
Test identification:	Test 8

Beam size :	305 × 165 UB 54
Column size :	254 × 254 UC 132
End-plate :	440mm × 240mm × 20mm

Major parameters :	Ct=60mm	Pt=120mm	Pit=0mm
	Cc+Pc=85mm	Pc=-	Pic=0mm
	Lb=295mm	G=120mm	Pi=175mm

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	22mm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 25 kN

Failure moment: (Maximum recorded moment 190.0 KNm)
Failure mode : Failure stage not reached

Remarks: 1) End-plate extended on tension side only
2) Pure moment loading

Table C105 : Moment-Rotation Data : Tong - Test 9
Connection Type : Extended end-plate connection to stiffened column

Tested by : Tong, C. S. [4.37]
Test identification: Test 9

Beam size :	305 × 165 UB 54
Column size :	254 × 254 UC 132
End-plate :	440mm × 240mm × 15mm

Major parameters :	Ct=60mm	Pt=120mm	Pit=0mm
	Cc+Pc=85mm	Pc=-	Pic=0mm
	Lb=295mm	G=120mm	Pi=175mm

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	22mm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 25 kN

Failure moment: 210.0 kNm
Failure mode : Inner bolt row fracture

Remarks: 1) End-plate extended on tension side only
2) Shear + moment loading

Table C105 : Moment-Rotation Data : Tong - Test 9

Moment (kNm)	Rotation (Radians)
0.00	0.00000
22.90	0.00060
36.80	0.00100
50.75	0.00140
61.75	0.00190
75.65	0.00250
88.60	0.00320
100.00	0.00400
115.50	0.00500
130.40	0.00600
143.30	0.00690
157.75	0.00870
169.20	0.01080
178.65	0.01250
182.15	0.01420
189.15	0.01750
195.10	0.02130
200.00	0.02770

Table C106 : Moment-Rotation Data : Tong - Test 10
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Tong, C. S. [4.37]
Test identification:	Test 10

Beam size :	305 × 165 UB 54
Column size :	254 × 254 UC 132
End-plate :	440mm × 240mm × 20mm

Major parameters :	Ct=60mm	Pt=120mm	Pit=0mm
	Cc+Pc=85mm	Pc=-	Pic=0mm
	Lb=295mm	G=120mm	Pi=175mm

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	22mm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 25 kN

Failure moment: 227.7 kNm
Failure mode : Inner bolt row fracture

Remarks: 1) End-plate extended on tension side only
2) Shear + moment loading

Table C106 : Moment-Rotation Data : Tong - Test 10

Moment (kNm)	Rotation (Radians)
0.00	0.00000
36.35	0.00060
50.25	0.00100
61.70	0.00160
74.65	0.00180
88.60	0.00230
100.55	0.00260
114.45	0.00940
131.40	0.00380
144.30	0.00450
157.25	0.00520
169.20	0.0060
176.70	0.00660
182.15	0.00690
187.65	0.00740
195.10	0.00830
200.00	0.00910
204.60	0.01020
213.00	0.01140
221.95	0.01310
225.45	0.01520

Table C107 : Moment-Rotation Data : Tong - Test 11
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Tong, C. S. [4.37]
Test identification:	Test 11

Beam size :	305 × 165 UB 54
Column size :	254 × 254 UC 73
End-plate :	440mm × 240mm × 25mm

Major parameters :	Ct=60mm	Pt=120mm	Pit=0mm
	Cc+Pc=85mm	Pc=-	Pic=0mm
	Lb=295mm	G=120mm	Pi=175mm

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	22mm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 25 kN

Failure moment: (Maximum recorded moment 110.0 kNm)
Failure mode : Failure stage not reached

Remarks: 1) End-plate extended on tension side only
2) Pure moment loading

Table C107 : Moment-Rotation Data : Tong - Test 11

Moment (kNm)	Rotation (Radians)
0.00	0.00000
20.05	0.00050
30.10	0.00060
40.40	0.00110
50.40	0.00130
60.70	0.00170
69.75	0.00190
80.50	0.00230
90.30	0.00260
99.85	0.00300
109.85	0.00340

Table C108 : Moment-Rotation Data : Tong - Test 13 Connection Type : Extended end-plate connection to stiffened column

Tested by :	Tong, C. S. [4.37]
Test identification:	Test 13

Beam size :	305 × 165 UB 54
Column size :	254 × 254 UC 132
End-plate :	425mm × 200mm × 12mm

Major parameters :	Ct=50mm	Pt=115mm	Pit=0mm
	Cc+Pc=85mm	Pc=-	Pic=0mm
	Lb=290mm	G=100mm	Pi=175mm

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	22mm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	300	-

Average pretension force of bolts : 25 kN

Failure moment: (Maximum recorded moment 138.5 kNm)
Failure mode : Failure stage not reached

Remarks: 1) End-plate extended on tension side only
2) Pure moment loading

Table C108 : Moment-Rotation Data : Tong - Test 13

Moment (kNm)	Rotation (Radians)
0.00	0.00000
22.00	0.00020
32.50	0.00060
43.00	0.00080
54.50	0.00140
65.00	0.00200
76.30	0.00240
86.20	0.00320
96.70	0.00420
107.50	0.00490
116.30	0.00650
122.80	0.00820
127.85	0.01000
133.30	0.01170
140.00	0.01300

Table C109 : Moment-Rotation Data : Tong - Test 14
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Tong, C. S. [4.37]
Test identification:	Test 14

Beam size :	305 × 165 UB 54
Column size :	254 × 254 UC 132
End-plate :	440mm × 240mm × 25mm

Major parameters :	Ct=60mm	Pt=120mm	Pit=0mm
	Cc+Pc=85mm	Pc=-	Pic=0mm
	Lb=295mm	G=120m	Pi=175mm

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	22mm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 25 kN

Failure moment: 267.5 kNm
Failure mode : Inner bolt row fracture

Remarks: 1) End-plate extended on tension side only
2) Pure moment loading

Table C109 : Moment-Rotation Data : Tong - Test 14

Moment (kNm)	Rotation (Radians)
0.00	0.00000
29.90	0.00050
53.75	0.00060
76.65	0.00110
97.54	0.00140
117.95	0.00180
139.34	0.00230
160.25	0.00280
180.65	0.00370
201.05	0.00450
220.95	0.00640
240.00	0.00860
250.85	0.01100
260.77	0.01490
266.74	0.01830

Table C110 : Moment-Rotation Data : Tong - Test 15
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Tong, C. S. [4.37]
Test identification:	Test 15

Beam size :	305 × 165 UB 54
Column size :	254 × 254 UC 132
End-plate :	440mm × 240mm × 12mm

Major parameters :	Ct=60mm	Pt=120mm	Pit=0mm
	Cc+Pc=85mm	Pc=-	Pic=0mm
	Lb=295mm	G=120mm	Pi=175mm

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	22mm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	300	-

Average pretension force of bolts : 135 kN

Failure moment: 185.5 kNm
Failure mode : End-plate

Remarks: 1) End-plate extended on tension side only
2) Pure moment loading

Table C110 : Moment-Rotation Data : Tong - Test 15

Moment (kNm)	Rotation (Radians)
0.00	0.00000
11.50	0.00010
22.00	0.00020
32.50	0.00030
42.50	0.00040
52.00	0.00060
63.00	0.00070
74.00	0.00100
85.00	0.00130
97.00	0.00150
105.00	0.00210
116.00	0.00320
127.00	0.00540
132.50	0.00720
138.50	0.00930
144.00	0.01170
150.00	0.00150
154.50	0.01840
160.00	0.02230
164.00	0.02750
169.00	0.03670

Table C111 : Moment-Rotation Data : Tong - Test 18
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Tong, C. S. [4.37]
Test identification:	Test 18

Beam size :	305 × 165 UB 54
Column size :	254 × 254 UC 132
End-plate :	440mm × 240mm × 20mm

Major parameters :	Ct=60mm	Pt=120mm	Pit=0mm
	Cc+Pc=85mm	Pc=-	Pic=0mm
	Lb=295mm	G=120mm	Pi=175mm

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	22mm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 135 kN

Failure moment: 243.0 kNm
Failure mode : Inner bolt row fracture

Remarks: 1) End-plate extended on tension side only
2) Pure moment loading

Table C111 : Moment-Rotation Data : Tong - Test 18

Moment (kNm)	Rotation (Radians)
0.00	0.00000
22.90	0.00010
34.85	0.00020
43.80	0.00030
54.75	0.00035
64.20	0.00050
76.63	0.00060
86.58	0.00070
97.55	0.00080
108.48	0.00090
117.93	0.00100
129.37	0.00120
139.82	0.00140
151.25	0.00160
161.72	0.00185
171.67	0.00230
183.61	0.00280
193.06	0.00390
199.53	0.00450
203.51	0.00520
209.30	0.00630
213.96	0.00760
220.93	0.00880
223.92	0.01040
231.87	0.01390
233.87	0.01710

Table C112 : Moment-Rotation Data : Zandonini et al - Test EP1-1
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Zandonini, R. and Zanon, P. [4.2]
Test identification:	Test EP1-1

Beam size :	IPE 300
Column size :	Rigid counterbeam
End-plate :	420mm × 180mm × 12mm

Major parameters :	Ct=-	Pt=120mm	Pit=0mm
	Cc=-	Pc=-	Pic=0mm
	Lb=300mm	G=105mm	Pi=180mm

Stiffeners :	Column web stiffeners
Beam fastening :	Weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	21.5mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	252.5	405.5
Beam web:	282.0	425.0
Column flange:	-	-
Column web:	-	-
End-plate:	321.0	449.0

Average pretension force of bolts : 40

Failure moment: 173.3 kNm
Failure mode : Failure of bolt in tension

Remarks:
1) End-plate extended on tension side only
2) Specimen subjected to loading and unloading cycles (no reversal loading)

Table C112 : Moment-Rotation Data : Zandonini et al - Test EP1-1

Moment (kNm)	Rotation (Radians)
0.00	0.00000
95.10	0.00145
103.50	0.00248
112.60	0.00537
121.30	0.00868
127.70	0.01280
138.00	0.01900
146.60	0.02645
155.20	0.03305
163.60	0.04050
173.80	0.05124
180.00	0.06200
185.40	0.07400
183.00	0.08140

Table C113 : Moment-Rotation Data : Zandonini et al - Test EP1-2
Connection Type : Extended end-plate connection to stiffened column

Tested by : Zandonini, R. and Zanon, P. [4.2]
Test identification: Test EP1-2

Beam size : IPE 300
Column size : Rigid counterbeam
End-plate : 420mm × 180mm × 15mm

Major parameters :	Ct=-	Pt=120mm	Pit=0mm
	Cc=-	Pc=-	Pic=0mm
	Lb=300mm	G=105mm	Pi=180mm

Stiffeners : Column web stiffeners
Beam fastening : Weld
Column fastening : 6 × M20 Grade 8.8 bolts
Hole type : 21.5mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	252.5	405.5
Beam web:	282.0	425.0
Column flange:	-	-
Column web:	-	-
End-plate:	300.0	460.0

Average pretension force of bolts : 40

Failure moment: 199.5 kNm
Failure mode : Failure of bolt in tension

Remarks: 1) End-plate extended on tension side only
2) Specimen subjected to loading and unloading cycles (no reversal loading)

Table C113 : Moment-Rotation Data : Zandonini et al - Test EP1-2

Moment (kNm)	Rotation (Radians)
0.00	0.00000
90.00	0.00080
110.00	0.00165
130.00	0.00413
140.00	0.00496
150.00	0.00826
160.00	0.01300
170.00	0.01900
180.00	0.02562
190.00	0.03388
195.50	0.04000
198.50	0.04793
200.00	0.05454
197.00	0.06000
190.00	0.06610

Table C113 : Moment-Rotation Data : Zandonini et al - Test EP1-3
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Zandonini, R. and Zanon, P. [4.2]
Test identification:	Test EP1-3

Beam size :	IPE 300
Column size :	Rigid counterbeam
End-plate :	420mm × 180mm × 18mm

Major parameters :	Ct=-	Pt=120mm	Pit=0mm
	Cc=-	Pc=-	Pic=0mm
	Lb=300mm	G=105mm	Pi=180mm

Stiffeners :	Column web stiffeners
Beam fastening :	Weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	21.5mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	252.5	405.5
Beam web:	282.0	425.0
Column flange:	-	-
Column web:	-	-
End-plate:	292.0	419.0

Average pretension force of bolts : 40

Failure moment:	204.4 kNm
Failure mode :	Failure of the end-plate

Remarks: 1) End-plate extended on tension side only
2) Specimen subjected to loading and unloading cycles (no reversal loading)

Table C114 : Moment-Rotation Data : Zandonini et al - Test EP1-3

Moment (kNm)	Rotation (Radians)
0.00	0.00000
110.00	0.00080
120.00	0.00125
130.50	0.00165
138.80	0.00205
149.00	0.00340
160.00	0.00458
175.00	0.00770
188.30	0.01290
199.00	0.01830
205.50	0.02460
203.70	0.02605

Table C115 : Moment-Rotation Data : Zandonini et al - Test EP1-4
Connection Type : Extended end-plate connection to stiffened column

Tested by : Zandonini, R. and Zanon, P. [4.2]
Test identification: Test EP1-4

Beam size :	IPE 300
Column size :	Rigid counterbeam
End-plate :	420mm × 180mm × 22mm

Major parameters :	Ct=-	Pt=120mm	Pit=0mm
	Cc=-	Pc=-	Pic=0mm
	Lb=300mm	G=105mm	Pi=180mm

Stiffeners :	Column web stiffeners
Beam fastening :	Weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	21.5mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	252.5	405.5
Beam web:	282.0	425.0
Column flange:	-	-
Column web:	-	-
End-plate:	221.0	354.0

Average pretension force of bolts : 40

Failure moment: 222.4 kNm
Failure mode : Failure of the end-plate

Remarks: 1) End-plate extended on tension side only
2) Specimen subjected to loading and unloading cycles (no reversal loading)

Table C115 : Moment-Rotation Data : Zandonini et al - Test EP1-4

Moment (kNm)	Rotation (Radians)
0.00	0.00000
100.00	0.00080
120.00	0.00125
140.00	0.00185
150.00	0.00250
160.30	0.00290
170.00	0.00375
180.00	0.00520
190.00	0.00665
200.00	0.00854
210.00	0.01125
219.80	0.01665
223.70	0.02000
225.00	0.03665
222.70	0.04020

Table C116 : Moment-Rotation Data : Zandonini et al - Test EP1-5
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Zandonini, R. and Zanon, P. [4.2]
Test identification:	Test EP1-5

Beam size :	IPE 300
Column size :	Rigid counterbeam
End-plate :	420mm × 180mm × 25mm

Major parameters :	Ct=-	Pt=120mm	Pit=0mm
	Cc=-	Pc=-	Pic=0mm
	Lb=300mm	G=105mm	Pi=180mm

Stiffeners :	Column web stiffeners
Beam fastening :	Weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	21.5mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	252.5	405.5
Beam web:	282.0	425.0
Column flange:	-	-
Column web:	-	-
End-plate:	186.0	259.0

Average pretension force of bolts : 40

Failure moment: 225.7 kNm
Failure mode : Failure of bolt in tension

Remarks: 1) End-plate extended on tension side only
2) Specimen subjected to loading and unloading cycles (no reversal loading)

Table C116 : Moment-Rotation Data : Zandonini et al - Test EP1-5

Moment (kNm)	Rotation (Radians)
0.00	0.00000
120.00	0.00080
130.00	0.00104
140.00	0.00125
150.00	0.00187
160.00	0.00250
170.00	0.00292
180.00	0.00415
190.00	0.00520
200.00	0.00665
210.00	0.00998
220.00	0.01370
227.10	0.02330
225.20	0.03077
220.00	0.03700

Table C117 : Moment-Rotation Data : Zandonini et al - Test EPB1-1
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Zandonini, R. and Zanon, P. [4.2]
Test identification:	Test EPB1-1

Beam size :	IPE 300
Column size :	Rigid counterbeam
End-plate :	520mm × 180mm × 12mm

Major parameters :	Ct=-	Pt=120mm	Lb=420mm
	Cc=-	Pc=120mm	Li=180mm
	G=105mm		

Stiffeners :	Column web stiffeners
Beam fastening :	Weld
Column fastening :	8 × M20 Grade 8.8 bolts
Hole type :	21.5mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	250.0	403.5
Beam web:	288.0	430.0
Column flange:	-	-
Column web:	-	-
End-plate:	321.0	449.0

Average pretension force of bolts : 40

Failure moment: -
Failure mode : Premature failure

Remarks: 1) End-plate extended on tension and compression side
2) Specimen subjected to loading and unloading cycles (no reversal loading)

Table C117 : Moment-Rotation Data : Zandonini et al - Test EPB1-1

Moment (kNm)	Rotation (Radians)
0.00	0.00000
80.00	0.00085
90.00	0.00165
100.00	0.00292
110.00	0.00583
120.00	0.01083
123.20	0.01262
147.80	0.03895
153.10	0.04292
158.00	0.04750
163.20	0.05080
166.10	0.05458

Table C118 : Moment-Rotation Data : Zandonini et al - Test EPB1-2
Connection Type : Extended end-plate connection to stiffened column

Tested by : Zandonini, R. and Zanon, P. [4.2]
Test identification: Test EPB1-2

Beam size : IPE 300
Column size : Rigid counterbeam
End-plate : 520mm × 180mm × 15mm

Major parameters :	Ct=-	Pt=120mm	Lb=420mm
	Cc=-	Pc=120mm	Li=180mm
	G=105mm		

Stiffeners : Column web stiffeners
Beam fastening : Weld
Column fastening : 8 × M20 Grade 8.8 bolts
Hole type : 21.5mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	250.0	403.5
Beam web:	288.0	430.0
Column flange:	-	-
Column web:	-	-
End-plate:	300.0	460.0

Average pretension force of bolts : 40

Failure moment: 202.8 kNm
Failure mode : Failure of bolt in tension

Remarks: 1) End-plate extended on tension and compression side
2) Specimen subjected to loading and unloading cycles (no reversal loading)

Table C118 : Moment-Rotation Data : Zandonini et al - Test EPB1-2

Moment (kNm)	Rotation (Radians)
0.00	0.00000
90.00	0.00062
100.00	0.00140
110.00	0.00125
120.00	0.00167
130.00	0.00250
140.00	0.00333
150.00	0.00500
160.00	0.00750
170.00	0.01083
180.00	0.01667
185.20	0.02000
190.00	0.02312
195.70	0.02667
200.00	0.03125
202.90	0.03562
203.80	0.04000
203.20	0.04667
200.00	0.05250

Table C119 : Moment-Rotation Data : Zandonini et al - Test EPB1-3
Connection Type : Extended end-plate connection to stiffened column

Tested by : Zandonini, R. and Zanon, P. [4.2]
Test identification: Test EPB1-3

Beam size : IPE 300
Column size : Rigid counterbeam
End-plate : 520mm × 180mm × 18mm

Major parameters :	Ct=-	Pt=120mm	Lb=420mm
	Cc=-	Pc=120mm	Li=180mm
	G=105mm		

Stiffeners : Column web stiffeners
Beam fastening : Weld
Column fastening : 8 × M20 Grade 8.8 bolts
Hole type : 21.5mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	250.0	403.5
Beam web:	288.0	430.0
Column flange:	-	-
Column web:	-	-
End-plate:	292.0	419.0

Average pretension force of bolts : 40

Failure moment: 204.4 kNm
Failure mode : Failure of the end-plate

Remarks: 1) End-plate extended on tension and compression side
2) Specimen subjected to loading and unloading cycles (no reversal loading)

Table C119 : Moment-Rotation Data : Zandonini et al - Test EPB1-3

Moment (kNm)	Rotation (Radians)
0.00	0.00000
100.00	0.00103
110.00	0.00124
120.00	0.00145
130.00	0.00186
140.00	0.00207
150.00	0.00248
160.00	0.00351
170.00	0.00475
180.00	0.00826
190.00	0.01157
200.00	0.01570
206.00	0.02000
203.00	0.02314
198.50	0.02686

Table C120 : Moment-Rotation Data : Zandonini et al - Test EPB1-4
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Zandonini, R. and Zanon, P. [4.2]
Test identification:	Test EPB1-4

Beam size :	IPE 300
Column size :	Rigid counterbeam
End-plate :	520mm × 180mm × 22mm

Major parameters :	Ct=-	Pt=120mm	Lb=4200mm
	Cc=-	Pc=120mm	Li=180mm
	G=105mm		

Stiffeners :	Column web stiffeners
Beam fastening :	Weld
Column fastening :	8 × M20 Grade 8.8 bolts
Hole type :	21.5mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	250.0	403.5
Beam web:	288.0	430.0
Column flange:	-	-
Column web:	-	-
End-plate:	221.0	354.0

Average pretension force of bolts : 40

Failure moment: 219.1 kNm
Failure mode : Excessive beam deformation

Remarks: 1) End-plate extended on tension and compression side
2) Specimen subjected to loading and unloading cycles (no reversal loading)

Table C120 : Moment-Rotation Data : Zandonini et al - Test EPB1-4

Moment (kNm)	Rotation (Radians)
0.00	0.00000
100.00	0.00062
110.00	0.00083
120.00	0.00103
130.00	0.00165
140.00	0.00186
150.00	0.00227
160.00	0.00267
170.00	0.00330
180.00	0.00434
190.00	0.00537
200.00	0.00661
210.00	0.00992
220.00	0.01425
210.00	0.01653

Table C121 : Moment-Rotation Data : Zandonini et al - Test EPB1-5
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Zandonini, R. and Zanon, P. [4.2]
Test identification:	Test EPB1-5

Beam size :	IPE 300
Column size :	Rigid counterbeam
End-plate :	520mm × 180mm × 25mm

Major parameters :	Ct=-	Pt=120mm	Lb=420mm
	Cc=-	Pc=120mm	Li=180mm
	G=105mm		

Stiffeners :	Column web stiffeners
Beam fastening :	Weld
Column fastening :	8 × M20 Grade 8.8 bolts
Hole type :	21.5mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	250.0	403.5
Beam web:	288.0	430.0
Column flange:	-	-
Column web:	-	-
End-plate:	186.0	259.0

Average pretension force of bolts : 40

Failure moment:	217.5 kNm
Failure mode :	Excessive beam deformation

Remarks: 1) End-plate extended on tension and compression side
2) Specimen subjected to loading and unloading cycles (no reversal loading)

Table C121 : Moment-Rotation Data : Zandonini et al - Test EPB1-5

Moment (kNm)	Rotation (Radians)
0.00	0.00000
100.00	0.00042
110.00	0.00062
120.00	0.00104
130.00	0.00125
140.00	0.00146
150.00	0.00167
160.00	0.00208
170.00	0.00250
180.00	0.00292
190.00	0.00437
200.00	0.00562
210.00	0.00750
220.00	0.01167
226.00	0.01583
225.50	0.02000
220.00	0.02208

Table C122 : Moment-Rotation Data : Prescott - Test 22 Connection Type : Extended end-plate connection to stiffened column

Tested by :	Prescott, A. T. [4.1]
Test identification:	Test 22

Beam size :	254 × 146 UB 43
Column size :	203 × 203 UC 71
End-plate :	375mm × 170mm × 20mm

Major parameters :	Ct=50mm	Pt=115mm	Pit=0mm
	Cc+Pc=85mm	Pc=-	Pic=0mm
	Lb=240mm	G=100mm	Pi=125mm

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	22mm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 25 kN

Failure moment: 191.9 kNm
Failure mode : Bolt fracture

Remarks: 1) End-plate extended on tension side only
2) Pure moment loading

Table C122 : Moment-Rotation Data : Prescott - Test 22

Moment (kNm)	Rotation (Radians)
0.00	0.00000
21.15	0.00025
31.50	0.00050
41.85	0.00100
54.00	0.00155
63.45	0.00215
74.25	0.00305
82.80	0.00363
90.00	0.00423
98.55	0.00622
106.20	0.00580
117.00	0.00691
124.65	0.00785
133.20	0.00905
140.40	0.01055
149.40	0.01250
154.80	0.01450
160.20	0.01664
164.70	0.01882
167.40	0.02050
169.20	0.02273

Table C123 : Moment-Rotation Data : Prescott - Test 23
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Prescott, A. T. [4.1]
Test identification:	Test 23

Beam size :	254 × 146 UB 43
Column size :	203 × 203 UC 71
End-plate :	375mm × 170mm × 25mm

Major parameters :	Ct=50mm	Pt=115mm	Pit=0mm
	Cc+Pc=85mm	Pc=-	Pic=0mm
	Lb=240mm	G=100mm	Pi=125mm

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	22mm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 25 kN

Failure moment: (Maximum recorded moment 174.1 kNm)
Failure mode : Failure stage not reached

Remarks: 1) End-plate extended on tension side only
2) Pure moment loading

Table C123 : Moment-Rotation Data : Prescott - Test 23

Moment (kNm)	Rotation (Radians)
0.00	0.00000
21.60	0.00019
32.45	0.00050
43.25	0.00100
54.05	0.00142
64.90	0.00185
75.25	0.00227
85.60	0.00270
96.00	0.00325
106.30	0.00391
117.15	0.00493
128.00	0.00610
138.75	0.00745
149.50	0.00905
155.00	0.01010
160.40	0.01137

Table C124 : Moment-Rotation Data : Prescott - Test 27 Connection Type : Extended end-plate connection to stiffened column

Tested by :	Prescott, A. T. [4.1]
Test identification:	Test 27

Beam size :	254 × 146 UB 43
Column size :	203 × 203 UC 71
End-plate :	375mm × 170mm × 20mm

Major parameters :	Ct=50mm	Pt=115mm	Pit=0mm
	Cc+Pc=85mm	Pc=-	Pic=0mm
	Lb=240mm	G=100mm	Pi=125mm

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	22mm drilled holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 100 kN

Failure moment: 187.3 kNm
Failure mode : Bolt fracture

Remarks: 1) End-plate extended on tension side only
2) Pure moment loading

Table C124 : Moment-Rotation Data : Prescott - Test 27

Moment (kNm)	Rotation (Radians)
0.00	0.00000
21.50	0.00020
37.50	0.00040
53.60	0.00070
63.90	0.00107
74.10	0.00125
84.90	0.00180
96.50	0.00240
124.10	0.00315
117.00	0.00405
126.80	0.00537
132.20	0.00650
137.50	0.00810
142.90	0.01035
148.30	0.01250
153.60	0.01485
155.40	0.01570

Table C125 : Moment-Rotation Data : Prescott - Test 28
Connection Type : Extended end-plate connection to stiffened column

Tested by :	Prescott, A. T. [4.1]
Test identification:	Test 28

Beam size :	254 × 146 UB 43
Column size :	203 × 203 UC 71
End-plate :	375mm × 170mm × 25mm

Major parameters :	Ct=50mm	Pt=115mm	Pit=0mm
	Cc+Pc=85mm	Pc=-	Pic=0mm
	Lb=240mm	G=100mm	Pi=125mm

Stiffeners :	Column web stiffeners opposite both beam flanges
Beam fastening :	10mm fillet weld
Column fastening :	6 × M20 Grade 8.8 bolts
Hole type :	22mm drilled holes

Measured Material Properties:
All sections grade 43A

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : 100 kN

Failure moment: 200.3 kNm
Failure mode : Bolt fracture

Remarks: 1) End-plate extended on tension side only
2) Pure moment loading

Table C126: Moment-Rotation Data : Prescott - Test 28

Moment (kNm)	Rotation (Radians)
0.00	0.00000
16.20	0.00025
21.60	0.00045
32.45	0.00073
43.25	0.00100
54.05	0.00130
64.90	0.00165
75.70	0.00190
85.60	0.00253
96.00	0.00315
106.30	0.00385
118.00	0.00455
123.00	0.00486
133.80	0.00562
144.15	0.00657
149.55	0.00758
155.00	0.00878
158.60	0.01015
160.00	0.01153

Table C127 : Moment-Rotation Data : Davison - Test JT/13 Connection Type : Extended end-plate connection to stiffened column

Tested by :	Davison, J. B. [4.41]
Test identification:	Test JT/13

Beam size :	254 × 102 UB 22
Column size :	152 × 152 UC 23
End-plate :	350mm × 135mm × 15mm

Major parameters :	Ct=40mm	Pt=100mm	Pit=0mm
	Cc+Pc=50mm	Pc=-	Pic=0mm
	Lb=260mm	G=76mm	Pi=160mm

Stiffeners :	3 column web stiffeners in tension zone and one in compression zone
Beam fastening :	4mm fillet weld all round
Column fastening :	6 × M16 Grade 8.8 bolts
Hole type :	16mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	292.35	-
Beam web:	297.06	-
Column flange:	266.26	-
Column web:	270.90	-
End-plate:	437.44	-

Average pretension force of bolts : Torque of about 160 Nm

Failure moment: (Maximum recorded moment 53.5 kNm right hand side beam and 49.9 kNm left hand side beam)

Failure mode : Test stopped before failure stage

Remarks:
1) End-plate extended on tension side only
2) Pure moment loading
3) Tabulated moment-rotation data is given by overlaying the results of Test JT/13b on those in Test JT/13

Table C126 : Moment-Rotation Data : Davison - Test JT/13

Moment (kNm)	Rotation (Radians)
0.00	0.00000
2.50	0.00010
7.25	0.00030
13.50	0.00070
17.50	0.00114
24.00	0.00185
30.25	0.00282
40.00	0.00433
46.00	0.00672
47.00	0.00737
48.25	0.00823
49.25	0.00895
50.50	0.00975
51.75	0.01083
58.75	0.01235
55.00	0.01343
56.50	0.01463
57.50	0.01582

Table C127 : Moment-Rotation Data : Davison - Test JT/13b Connection Type : Extended end-plate connection to stiffened column

Tested by :	Davison, J. B. [4.41]
Test identification:	Test JT/13b

Beam size :	254 × 102 UB 22
Column size :	152 × 152 UC 23
End-plate :	350mm × 135mm × 15mm

Major parameters :	Ct=40mm	Pt=100mm	Pit=0mm
	Cc+Pc=50mm	Pc=-	Pic=0mm
	Lb=260mm	G=76mm	Pi=160mm

Stiffeners :	3 column web stiffeners in tension zone and one in compression zone
Beam fastening :	4mm fillet weld all round
Column fastening :	6 × M16 Grade 8.8 bolts
Hole type :	16mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	292.35	-
Beam web:	297.06	-
Column flange:	266.26	-
Column web:	270.90	-
End-plate:	437.44	-

Average pretension force of bolts : Torque of about 150 Nm

Failure moment: -

Failure mode : Weld failure

Remarks: 1) End-plate extended on tension side only
2) Pure moment loading
3)Tabulated moment-rotation data is given by overlaying the results of Test JT/13b on those in Test JT/13
4) Re-test of Test JT/13

Table C127 : Moment-Rotation Data : Davison - Test JT/13b

Moment (kNm)	Rotation (Radians)
0.00	0.0000
2.50	0.00010
7.25	0.00030
13.50	0.00070
17.50	0.00114
24.00	0.00185
30.25	0.00282
40.00	0.00433
46.00	0.00672
47.00	0.00737
48.25	0.00823
49.25	0.00895
50.50	0.00975
51.75	0.01083
58.75	0.01235
55.00	0.01343
56.50	0.01463
57.50	0.01582

Table C128 : Moment-Rotation Data : Davison - Test CT/04 Connection Type : Extended end-plate connection to stiffened column

Tested by : Davison, J. B. [4.41]
Test identification: Test CT/04

Beam size : 254 × 102 UB 22
Column size : 152 × 152 UC 23
End-plate : 350mm × 135mm × 15mm

Major parameters :	Ct=40mm	Pt=100mm	Pit=0mm
	Cc+Pc=50mm	Pc=-	Pic=0mm
	Lb=260mm	G=76mm	Pi=160mm

Stiffeners : 3 column web stiffeners in tension zone and one in compression zone
Beam fastening : 4mm fillet weld all round
Column fastening : 6 × M16 Grade 8.8 bolts
Hole type : 16mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	287.80 left (289.20 right)	-
Beam web:	301.13 left (303.78 right)	-
Column flange:	276.29	-
Column web:	275.47	-
End-plate:	266.01	-

Average pretension force of bolts : Torque of about 150 Nm

Failure moment: (Maximum recorded moment 57.5 kNm right hand side and 58.1 kNm left hand side)
Failure mode : Test stopped due to beams were biginning to fail by lateral torsional instability

Remarks:
1) End-plate extended on tension side only
2) Pure moment loading
3) End-plate distored by 15mm
4) Tabulated moment-rotation data corresponds to average of left and right hand beam results

Table C128: Moment-Rotation Data : Davison - Test CT/04

Moment (kNm)	Rotation (Radians)
0.00	0.00000
6.00	0.00025
10.75	0.00050
16.50	0.00100
21.75	0.00140
27.25	0.00185
33.50	0.00250
37.50	0.00350
39.75	0.00370
42.50	0.00475
44.50	0.00580
46.50	0.00740
49.00	0.00920
50.25	0.01125
52.00	0.01265
53.50	0.01440
54.50	0.01560
56.00	0.01665
56.50	0.01770
57.50	0.01900

Table C129 : Moment-Rotation Data : Davison - Test CT/06 Connection Type : Extended end-plate connection to stiffened column

Tested by : Davison, J. B. [4.41]
Test identification: Test CT/06

Beam size : 254 × 102 UB 22
Column size : 152 × 152 UC 23
End-plate : 350mm × 135mm × 15mm

Major parameters :	Ct=40mm	Pt=100mm	Pit=0mm
	Cc+Pc=50mm	Pc=-	Pic=0mm
	Lb=260mm	G=76mm	Pi=160mm

Stiffeners : 3 column web stiffeners in tension zone and one in compression zone
Beam fastening : 4mm fillet weld all round
Column fastening : 6 × M16 Grade 8.8 bolts
Hole type : 16mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	287.80 left (292.35 right)	-
Beam web:	301.13 left (297.06 right)	-
Column flange:	276.29	-
Column web:	275.47	-
End-plate:	266.01	-

Average pretension force of bolts : Torque of about 150 Nm

Failure moment: (Maximum recorded moment 57.5 kNm right hand side and 58.1 kNm left hand side)
Failure mode : Test stopped due to beams were biginning to fail by lateral torsional instability

Remarks:
1) End-plate extended on tension side only
2) Pure moment loading
3) End-plate distored by 15mm
4) Tabulated moment-rotation data corresponds to average of left and right hand beam results

Table C129 : Moment-Rotation Data : Davison - Test CT/06

Moment (kNm)	Rotation (Radians)
0.00	0.00000
13.00	0.00025
21.00	0.00040
28.50	0.00070
35.00	0.00100
41.50	0.00180
46.00	0.00260
49.50	0.00380
52.00	0.00535
54.50	0.00680
56.50	0.00830
58.50	0.01000
60.00	0.01220
62.50	0.01400
64.50	0.01600
65.25	0.01750
66.00	0.01860
67.50	0.01980

Table C130 : Moment-Rotation Data : Chakrabarti -Test 1 and 2 Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Chakrabarti, B. [4.42]
Test identification:	Test 1 and 2

Beam size :	254 × 102 UB 28
Column size :	152 × 152 UC 37
End-plate :	385mm × 154mm × 15mm

Major parameters :	Ct=60mm	Pt=110mm	Pit=0mm
	Cc+Pc=80mm	Pc=-	Pic=0mm
	Lb=245mm	G=100mm	Pi=135mm

Stiffeners :	No
Beam fastening :	10mm fillet weld all round
Column fastening :	6 × M16 Grade 8.8 bolts
Hole type :	18mm oversize holes

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	-	-
Beam web:	-	-
Column flange:	-	-
Column web:	-	-
End-plate:	-	-

Average pretension force of bolts : Hand tightened nuts and bolts

Failure moment: 96 kNm

Failure mode : Excessive yielding of column flanges and compression in the column web

Remarks:
1) End-plate extended on tension side only
2) Pure moment loading
3) Tests 1 and 2 correspond respectively to left and right side of the cruciform type test
3) Moment-rotation data corresponds to average results of right and left hand side connections

Table C130 : Moment-Rotation Data : Chakrabarti -Test 1

Moment (kNm)	Rotation (Radians)
0.00	0.00000
5.00	0.00055
10.00	0.00080
15.00	0.00150
20.00	0.00176
25.00	0.00227
30.00	0.00272
35.00	0.00336
40.00	0.00400
45.00	0.00500
50.00	0.00720
56.70	0.01000
60.00	0.01165
65.40	0.01500
71.80	0.02000
76.00	0.02500
79.20	0.03000
82.40	0.03500
84.80	0.04000
87.00	0.04500
89.80	0.05000
91.60	0.05500
93.40	0.06000
94.00	0.06500
95.00	0.07000
95 0.07720	

Table C131 : Moment-Rotation Data : Janss et al. Test 01
Connection Type : Extended end-plate connection to unstiffened column

Tested by : Janss,J. ;Jaspart,J.P. and Maquoi,R. [4.43]
Test identification: Test 01

Beam size : IPE 200
Column size : HEB160
End-plate : 270mm × 140mm × 15mm

Major parameters :			
	Ct=25mm	Pt=80mm	Pit=0mm
	Cc+Pc=55mm	Pc=-	Pic=0mm
	Lb=190mm	G=90mm	Pi=110mm

Stiffeners : No
Beam fastening : Weld
Column fastening : 6 × M18 Grade 10.9 HS bolts
Hole type : NO clearance

		Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Measured Material Properties:	Beam flange:	351.0	456.0
	Beam web:	371.0	477.0
	Column flange:	280.0	422.3
	Column web:	298.8	422.0
	End-plate:	370.0	556.0

Average pretension force of bolts : preloaded to 0.8 times the material yield stress

Failure moment: 79.68 kNm
Failure mode : Test discontinued due to damage to hydraulic ram

Remarks: 1) End-plate extended on tension side only
2) 300 kN axial load on column
3) Static loading

Table C131 : Moment-Rotation Data : Janss et al. Test 01

Moment (kNm)	Rotation (Radians)
0.00	0.00000
10.00	0.00025
20.00	0.00085
24.75	0.00130
30.00	0.00150
34.75	0.00170
40.00	0.00215
45.25	0.00260
50.00	0.00345
55.25	0.00455
60.00	0.00520
64.20	0.01085
66.30	0.01390
68.95	0.01650
70.55	0.01910

Table C132 : Moment-Rotation Data : Janss et al. Test 04
Connection Type : Extended end-plate connection to unstiffened column

Tested by : Janss,J. ;Jaspart,J.P. and Maquoi,R. [4.43]
Test identification: Test 04

Beam size : IPE 200
Column size : HEB 160
End-plate : 270mm × 140mm × 15mm

Major parameters :	Ct=25mm	Pt=80mm	Pit=0mm
	Cc+Pc=55mm	Pc=-	Pic=0mm
	Lb=190mm	G=90mm	Pi=110mm

Stiffeners : No
Beam fastening : Weld
Column fastening : 6 × M18 Grade 10.9 HS bolts
Hole type : NO clearance

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	351.0	456.0
Beam web:	371.0	477.0
Column flange:	280.0	422.3
Column web:	298.8	422.0
End-plate:	370.0	556.0

Average pretension force of bolts : preloaded to 0.8 times the material yield stress

Failure moment: 75.14 kNm
Failure mode : Plasticity of the column flange

Remarks: 1) End-plate extended on tension side only
2) 700 kN axial load on column
3) Static loading

Table C132 : Moment-Rotation Data : Janss et al. Test 04

Moment (kNm)	Rotation (Radians)
0.00	0.00000
10.00	0.00085
15.80	0.00110
20.00	0.00130
14.75	0.00175
30.00	0.00195
35.80	0.00240
40.00	0.00260
45.25	0.00304
50.00	0.00390
55.75	0.00695
57.40	0.01045
60.00	0.01390
63.70	0.01740
66.30	0.02000
68.40	0.02500
70.00	0.02935
73.95	0.03565
75.15	0.04000

Table C133 : Moment-Rotation Data : Janss et al. Test 07 Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Janss,J. ;Jaspart,J.P. and Maquoi,R. [4.43]
Test identification:	Test 07

Beam size :	IPE 200
Column size :	HEB 160
End-plate :	270mm × 140mm × 15mm

Major parameters :	Ct=25mm	Pt=80mm	Pit=0mm
	Cc+Pc=55mm	Pc=-	Pic=0mm
	Lb=190mm	G=90mm	Pi=110mm

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	6 × M18 Grade 10.9 HS bolts
Hole type :	NO clearance

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	351.0	456.0
Beam web:	371.0	477.0
Column flange:	280.0	422.3
Column web:	298.8	422.0
End-plate:	370.0	556.0

Average pretension force of bolts : preloaded to 0.8 times the material yield stress

Failure moment: 81.19 kNm
Failure mode : End-plate failure and plasticity of the column web

Remarks: 1) End-plate extended on tension side only
2) Static loading

Table C133 : Moment-Rotation Data : Janss et al. Test 07

Moment (kNm)	Rotation (Radians)
0.00	0.00000
10.00	0.00020
20.00	0.00040
25.25	0.00065
30.00	0.00085
33.70	0.00110
40.00	0.00130
44.20	0.00130
50.00	0.00285
56.30	0.00455
60.00	0.00610
65.25	0.01220
67.90	0.01590
70.00	0.02000
73.70	0.02695
75.80	0.03220
78.95	0.03655
80.00	0.04130
81.19	0.04300

Table C134 : Moment-Rotation Data : Janss et al. Test 010
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Janss,J. ;Jaspart,J.P. and Maquoi,R. [4.43]
Test identification:	Test 010

Beam size :	IPE 300
Column size :	HEB 160
End-plate :	380mm × 150mm × 20mm

Major parameters :	Ct=30mm	Pt=90mm	Pit=0mm
	Cc+Pc=60mm	Pc=-	Pic=0mm
	Lb=290mm	G=90mm	Pi=200mm

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	6 × M22 Grade 10.9 HS bolts
Hole type :	NO clearance

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	303.0	447.0
Beam web:	314.0	443.0
Column flange:	280.0	422.3
Column web:	298.8	422.0
End-plate:	297.0	463.4

Average pretension force of bolts : preloaded to 0.8 times the material yield stress

Failure moment: 124.58 kNm
Failure mode : Test stopped to avoid bolt fracture

Remarks:
1) End-plate extended on tension side only
2) Static loading
3) Washer under bolt nut

Table C134 : Moment-Rotation Data : Janss et al. Test 010

Moment (kNm)	Rotation (Radians)
0.00	0.00000
10.00	0.00085
20.00	0.00085
30.00	0.00085
40.00	0.00110
50.00	0.00120
56.00	0.00130
60.00	0.00130
65.60	0.00155
70.00	0.00175
76.10	0.00125
80.00	0.00240
86.10	0.00260
90.00	0.00305
96.15	0.00350
100.00	0.00435
105.60	0.00455
110.00	0.00520
115.20	0.00610
120.00	0.00785
124.60	0.01000

Table C135 : Moment-Rotation Data : Janss et al. Test 013
Connection Type : Extended end-plate connection to unstiffened column

Tested by :	Janss,J. ;Jaspart,J.P. and Maquoi,R. [4.43]
Test identification:	Test 013

Beam size :	IPE 200
Column size :	HEB 240
End-plate :	270mm × 120mm × 18mm

Major parameters :	Ct=25mm	Pt=80mm	Pit=0mm
	Cc+Pc=55mm	Pc=-	Pic=0mm
	Lb=190mm	G=72mm	Pi=110mm

Stiffeners :	No
Beam fastening :	Weld
Column fastening :	6 × M16 Grade 10.9 HS bolts
Hole type :	NO clearance

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	351.0	456.0
Beam web:	371.0	477.0
Column flange:	310.0	443.0
Column web:	349.5	458.5
End-plate:	293.0	428.5

Average pretension force of bolts : preloaded to 0.8 times the material yield stress

Failure moment: 62.7 kNm
Failure mode : Plasticity in the column web

Remarks: 1) End-plate extended on tension side only
2) Static loading

Table C135 : Moment-Rotation Data : Janss et al. Test 013

Moment (kNm)	Rotation (Radians)
0.00	0.00000
10.00	0.00175
14.75	0.00260
20.00	0.00300
25.25	0.00365
30.00	0.00410
34.75	0.00430
40.00	0.00450
46.30	0.00515
50.00	0.00690
53.70	0.01290
56.85	0.02000
60.00	0.02170
62.60	0.02730
60.80	0.03055
60.80	0.05000
60.80	0.08000
60.80	0.01045

Table C136 : Moment-Rotation Data : Janss et al. Test 014
Connection Type : Extended end-plate connection to unstiffened column

Tested by : Janss,J. ;Jaspart,J.P. and Maquoi,R. [4.43]
Test identification: Test 014

Beam size : IPE 200
Column size : IPE 300
End-plate : 270mm × 140mm × 15mm

Major parameters :	Ct=25mm	Pt=80mm	Pit=0mm
	Cc+Pc=55mm	Pc=-	Pic=0mm
	Lb=190mm	G=90mm	Pi=110mm

Stiffeners : No
Beam fastening : Weld
Column fastening : 6 × M18 Grade 10.9 HS bolts
Hole type : NO clearance

Measured Material Properties:

	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
Beam flange:	351.0	456.0
Beam web:	371.0	477.0
Column flange:	303.0	447.0
Column web:	314.0	443.0
End-plate:	370.0	556.1

Average pretension force of bolts : preloaded to 0.8 times the material yield stress

Failure moment: 83.63 kNm
Failure mode : Test stopped (not possible to increase the load)

Remarks: 1) End-plate extended on tension side only
2) Static loading
3) 2 washers under bolt nut

Table C136 : Moment-Rotation Data : Janss et al. Test 014

Moment (kNm)	Rotation (Radians)
0.00	0.00000
10.00	0.00002
20.00	0.00108
30.00	0.00175
40.00	0.00280
45.25	0.00710
50.00	0.01140
56.85	0.02000
60.00	0.02610
62.10	0.03280
65.25	0.04000
67.40	0.04705
70.00	0.05460
72.65	0.06000
74.75	0.07000
78.95	0.08000
80.00	0.08630
82.10	0.10000
82.10	0.13300